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UPGRADING WASTEWATER STABILIZATION PONDS TO MEET NEW DISCHARGE STANDARDS

UTAH WATER RESEARCH LABORATORY

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16. ABSTRACT

The question as to whether lagoons, as they now exist, meet the new secondary treatment standards and what methods would work to upgrade lagoon treatment in cases where they presently do not meet the standards is of high priority for many Regional Offices of EPA. The symposium was held for EPA staff and State Officials to review the Office of Research and Development's program for upgrading lagoons. The intermittent sand filter, land application of algae laden effluents and the submerged rock filter offer good potential for cost effective upgrading.

The basic biology of the treatment mechanism, disinfection technology, the controlled discharge operation (and results), cost effective analysis, and new fields of research were covered by speakers. Intermittent sand filtration, submerged rock filtration, and land application of sewage effluents are effective alternatives to removing algae from sewage lagoons.

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The great expenditure of time and effort made by the Symposium participants and authors of the papers is gratefully acknowledged. Without such willingness to share knowledge and experiences, meetings such as this would be impossible.

E. Joe Middlebrooks
Dean
College of Engineering
Utah State University

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TABLE OF CONTENTS

Page

INTRODUCTORY REMARKS
W. A. Rosenkranz 1

REVIEW OF EPA RESEARCH AND DEVELOPMENT LAGOON UPGRADING PROGRAM FOR FISCAL YEARS 1973, 1974, and 1975
R. F. Lewis 3

STATE OF THE ART OF LAGOON WASTEWATER TREATMENT
R. E. McKinney 15

CONSTRUCTION PROCEDURES AND REVIEW OF PLANS AND GRANT APPLICATIONS
W. R. Uhte 21

POLISHING LAGOON EFFLUENTS WITH SUBMERGED ROCK FILTERS
W. J. O'Brien 31

STABILIZATION POND UPGRADING WITH INTERMITTENT SAND FILTERS
E. J. Middlebrooks and G. R. Marshall 47

INTERMITTENT SAND FILTRATION TO UPGRADE LAGOON EFFLUENTS–PRELIMINARY REPORT
J. H. Reynolds, S. E. Harris, D. Hill, D. S. Filip, and E. J. Middlebrooks 71

PERFORMANCE OF RAW WASTE STABILIZATION LAGOONS IN MICHIGAN WITH LONG PERIOD STORAGE BEFORE DISCHARGE
D. M. Pierce 89

SEPARATION OF ALGAE CELLS FROM WASTEWATER LAGOON EFFLUENTS
R. A. Gearheart and E. J. Middlebrooks 137

UPGRADING LAGOON TREATMENT WITH LAND APPLICATION
R. E. Thomas 183

LAGOON EFFLUENT SOLIDS CONTROL BY BIOLOGICAL HARVESTING
W. R. Duffer 187

PROGRESS REPORT: BLUE SPRINGS LAGOON STUDY, BLUE SPRINGS, MISSOURI
C. M. Walter and S. L. Bugbee 191

COST-EFFECTIVENESS ANALYSIS FOR WATER POLLUTION CONTROL
R. Smith 199

NOTES

Research on Cold Climate Wastewater Lagoons
H. J. Coutts 227

Polishing Gravel and Activated Carbon Filter on Aerated Lagoon Effluents
G. Hartmann 229

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TABLE OF CONTENTS (Continued)

Page

MEMBERS OF THE SYMPOSIUM 231

APPENDIX 233

AUTHOR INDEX 237

SUBJECT INDEX 241

INTRODUCTORY REMARKS AT THE EPA/ORD INTRA-AGENCY WORKSHOP ON WASTEWATER TREATMENT PONDS

W. A. Rosenkranz[1]

In June of 1973, Region X sent a letter to consulting engineers, municipal officials, agencies, and others concerned with Water Pollution Control. That letter contained the following statement:

> "It is generally recognized that neither standard oxidation lagoons nor aerated lagoons by themselves will be able to achieve the required level of treatment. Therefore, this region will not be able to approve construction grants for projects proposing lagoons as the method of wastewater treatment unless such lagoons are part of a system designed to preclude any discharge or which incorporates supplemental treatment components capable of improving the effluent quality to an acceptable level. Meeting our minimum treatment requirements might be achieved by combining the lagoon treatment with an irrigation effluent disposal system, by adding supplemental treatment components or by constructing a non-overflow lagoon."

A report prepared for the Office of Research and Development by George Barsom of Ryckman, Edgerley, Tomlinson, and Associates, Inc., entitled "Lagoon Performance and the State of Lagoon Technology" had an overall negative viewpoint as to the suitability of lagoons for secondary treatment and repeatedly cited the lack of hard performance data on lagoons. The data contained in the report gave no assurance that there were existing lagoon designs capable of meeting the secondary treatment standards.

On behalf of the Office of Research and Development of the Environmental Protection Agency (EPA), I wish to welcome you to the Intra-Agency Workshop on Wastewater Treatment Pond Upgrading Technology. You are a select group of EPA staff and state officials brought together to review the Office of Research and Development's program for upgrading wastewater treatment ponds. Before you leave, we intend to pass on to you the most recent results of our on-going research programs, and program plan for the future. The results of the workshop should provide you with solutions to the problems posed by Region X and Mr. Barsom.

The question as to whether lagoons, as they now exist, meet the new secondary treatment standards and what methods would work to upgrade lagoon treatment in cases where they presently do not meet the standards is of high priority for many Regional Offices of EPA. This is because nearly 90 percent of the wastewater lagoons in the United States are located in small communities of 5,000 people or less. These communities, many with an average daily wastewater flow of only 175,000 to 200,000 gallons, do not have the resources to keep operators at the treatment sites throughout the day.

EPA, in my view, has a definite responsibility in this area. The agency has defined secondary treatment, obviously with the knowledge of the numbers of treatment ponds that exist, and knowing that effluent quality from the ponds is questionable. I also clearly recall that, in the early days of the construction grant program, the regions spent considerable effort in convincing state agencies that treatment ponds were a viable treatment process. If there can be developed economical upgrading techniques which can save the sunk capital costs, we owe that to the communities with ponds. A high degree of technical knowledge is usually lacking in the operators from these communities. Often, only periodic inspection or maintenance is carried out by the community's work force. Therefore, the development of relatively inexpensive methods for upgrading lagoons that do not require sophisticated and constant operation or extensive maintenance is urgently needed.

The majority of the research needs identified this year by our regional offices pertaining to biological treatment are concerned with lagoon upgrading, particularly algal removal from lagoon effluents. As you may be aware, three pilot-scale research contracts are currently in progress to evaluate removal of algae from lagoon effluents.

The projects and technology to be discussed during this seminar were planned to have a quick pay-off because it is felt that they can be completed in time to impact the July 1, 1977, federal deadline for achievement of secondary treatment in all municipal installations. The projects funded at Utah State University, which are the intermittent sand filter and land application of algae laden effluents, and at the University of Kansas, the submerged rock filter, offer good potential for cost effective upgrading technology.

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[1] W. A. Rosenkranz is Director, Municipal Pollution Control Division, Office of Research and Development, Environmental Protection Agency, Washington, D.C.

We feel that our research program plan will identify the inadequacies of facultative and aerated lagoons. Recommended practices for disinfecting lagoon effluents are needed, whether we are considering inherent destruction of coliforms (natural die-off relative to detention time), chlorination, or other disinfection processes. Work is also planned on control of nutrients. The broad objective is the development and demonstration of technology which can be recommended in your respective regions as cost effective for upgrading applications.

Prior to today's assembly, there were at least two major symposia held to disseminate information on wastewater treatment pond technology, and several EPA Technology Transfer Seminars to disseminate upgrading technology to consulting engineers. This conference differs from the make-up of previous seminars and symposia in that investigators engaged in research activities have been brought together with regulatory personnel and those engaged in approving treatment plant design to discuss current research, design, and operational findings of wastewater treatment ponds.

The workshop agenda contains topics including the basic biology of the treatment mechanism, algal removal process evaluations, disinfection technology, and cost effective analysis. We look forward to reviewing any special problem confronting your specific region. While we may not have all the answers to these problems, we must make our R&D program responsive to your needs. An assembly such as this is a useful mechanism to that end.

In the next two and one-half days, we will recap past experience and discuss on-going projects in detail. You will be informed of our R&D program plan, which was prepared to delineate the course of action and tentative resource allocations to produce an array of upgrading technologies. Unfortunately, our declining budget situation has precluded allocating resources as originally proposed, but we will attempt to optimize our investment to ensure maximum utilization of resources to produce the results required. Consequently, our demonstration sites will not span all temperature zones, waste compositions, etc., but sites representative of the broadest spectrum of conditions possible will be selected.

As a result of specific problems in Region X, personnel from the Advanced Waste Treatment Research Laboratory in Cincinnati, Ohio, and R&D Headquarters met with the regional staff to discuss specifics of their policy statement and the R&D program plan. Based on those conversations, it was concluded that the research program of improving effluents of wastewater treatment ponds would be responsive to Region X's needs. However, it was also concluded that the output from the program would be available in design-manual form too late. Region X staff urged us to prepare an interim report on whatever methodology was available. It was readily apparent that regional offices need whatever data are available as quickly as possible so that they can be better prepared to ensure that the requirements of the FWPCA Amendment of 1972 would be satisfied with regard to evaluation of cost effective alternatives.

Mr. John Rhett, Deputy Assistant Administrator for Water Program Operations, repeated the regions' request for interim data from our upgrading program. It became imperative that an interim report on R&D's program be prepared. We know of no better way to provide you with that interim report than to gather together as we are now doing. Your needs were the catalyst that precipitated the events leading up to this workshop.

As a result of your requests and input, we have adjusted our program plan. Due to reduced research budget resources, we are constrained in our participation in the construction phases of demonstration projects. We are in a position where we must rely on you to assist us in putting together demonstration projects with joint R&D-construction grant funding. Your assistance is required to optimize our R&D dollars. You will continue to have a direct hand in planning future research in this field. Either before you leave, or when you return to your respective regions, we want your recommendations for future development and/or demonstration projects.

Before closing I'd like to mention two other factors which will—or should—impact our R&D. This first is recognition of new technologies that may impact pond design and performance. Such things as pressure and vacuum sewer systems may change the volume and character of raw wastes. The other factor is that, while we are attempting to upgrade ponds to secondary levels, many are or will be located on water quality limited streams, requiring best practicable treatment above secondary. Let's not repeat our past mistakes and ignore future technology and/or administrative actions which could further impact the use of treatment ponds.

We intend to be directly responsive to your needs and hope that the information passed on to you during the course of the workshop will satisfy your most immediate requirements. Perhaps this meeting will set the stage for future meetings of similar nature. If you have suggestions for improving the substance or format we would like to have them.

REVIEW OF EPA RESEARCH AND DEVELOPMENT LAGOON UPGRADING PROGRAM FOR FISCAL YEARS 1973, 1974, AND 1975

R. F. Lewis[1]

Introduction

This paper will review the Environmental Protection Agency Research and Development Program for upgrading the performance of lagoon wastewater treatment systems. Specifically, those lagoon projects initiated in fiscal years 1973 and 1974 and those planned for fiscal year 1975 will be discussed.

In developing an upgrading program for any widely used wastewater treatment process, it is important to consider the types of communities served by that process, the personnel and financial resources of those communities, the performance norm and variability of the process operating in a conventional non-upgraded mode, and the degree of effluent quality improvement required to satisfy pertinent federal and water quality standards. Before reviewing the fiscal year 1973, 1974, and 1975 projects, these considerations will first be discussed briefly as related to facultative and aerated lagoons. The role these factors had in determining the course and priorities of the EPA research and development lagoon upgrading program will be emphasized.

Approximately 90 percent of the wastewater lagoons in the United States are located in small communities of 10,000 people or less. During the period of 1940-1974, wastewater lagoons rapidly gained popularity as a means of treating wastewaters from isolated industries, such as meat packing plants, and from small rural communities. A recent report by Barsom (1973) shows that in 1945 there were 45 lagoons treating municipal wastes, while by 1960 the number of lagoons had increased to 4,476. To this number must also be added the many privately owned lagoons treating wastewaters from motels, schools, trailer parks, and feed lots that have not been listed in state or national registers. The proliferation and acceptance of lagoon treatment for small communities was especially evident in the western and southern portions of the United States.

For these small communities, lagoons have proven to be relatively stable treatment systems able to handle fairly wide diurnal or daily fluctuations in wastewater flow and organic loading with negligible effect on effluent quality. The reason lagoons can handle these variations is that the long detention times utilized (10 to 150 days) provide great equalization of flow and load. Lagoons generally cost less than other biological processes to install, and they do not require around-the-clock surveillance. Maintenance work can be performed by the community work force.

The first lagoons to be used for secondary treatment were generally single-cell facultative lagoons. Due to the problems of short circuiting and poor treatment during cold weather with these early single-cell systems, multi-cell facultative lagoons, aerated lagoons, or combinations thereof are specified by most design engineers today. In certain states, intermittent-discharge lagoons have also been widely used in recent years.

Summary of Existing Lagoon Performance and Characteristic Effluent Quality

A comprehensive analysis of the performance capabilities of existing lagoons is difficult because of the lack of consistent, reliable information and analytical data. Since most lagoons are located in small communities that do not have highly trained personnel, very few laboratory analyses are performed on either the influent or effluent from these lagoons. The Barsom (1973) survey reported that only 28 of 50 states required routine monitoring of influent loading parameters. The problems most often cited by state engineers were offensive odors (all 50 states), short circuiting (23 states), and algae carryover problems (21 states).

One of the major obstacles in improving the application of lagoon technology in the past has been the failure of engineers to relate lagoon performance to causative factors, and to modify established design criteria accordingly. This has created a lack of confidence in the treatment technique in general, and a reluctance on the part of regulatory authorities to endorse new applications of lagoon treatment systems.

---

[1] R. F. Lewis is with the National Environmental Research Center, Advanced Waste Treatment Research Laboratory, Office of Research and Development, Cincinnati, Ohio.

The state-of-the-art report on lagoon technology by Barsom (1973) is the most comprehensive study of lagoon performance presently available. Information was assembled during this study by questionnaires and direct contact of state water pollution control agencies, municipalities, and independent researchers. The investigation evaluated data on lagoon performance from all 50 states and approximately 3,000 lagoon installations although Barsom felt that the data were reliable for only about 200 of the lagoons surveyed. Figure 1, taken from Barsom's report, shows the national average median effluent values for the BOD and suspended solids of facultative lagoons, aerated lagoons, oxidation ditches, and tertiary lagoons. The average median effluent BOD ranged from 23 to 42 mg/l and the average median effluent suspended solids ranged from 37 to 67 mg/l. Figures 2 and 3, also taken from Barsom's report, indicate the BOD and suspended solids levels in facultative lagoon effluents in different geographical areas of the United States, showing the average median in each area and the range of values found in that area. For these areas, the average median effluent BOD and suspended solids ranged from 25 to 75 mg/l and from 40 to 540 mg/l, respectively. For aerated lagoons, Figures 4 and 5, again taken from Barsom's report, show that the average median effluent BOD by geographic region varied from 30 to 80 mg/l, and the average median effluent suspended solids varied from 60 to 210 mg/l.

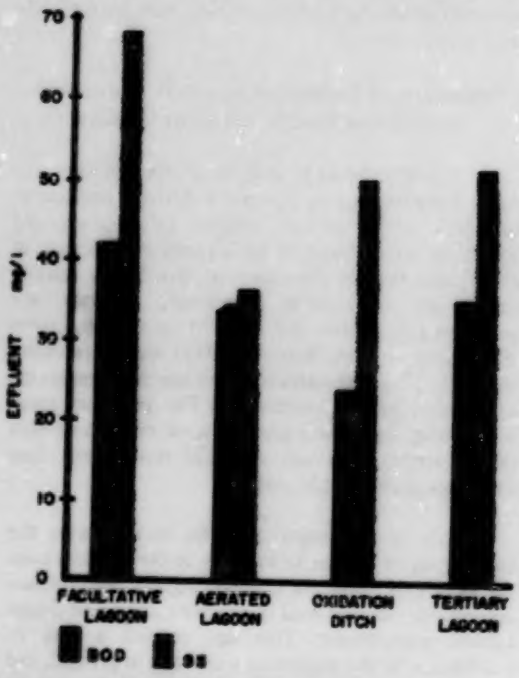

Figure 1. National average median effluent values for BOD and suspended solids.

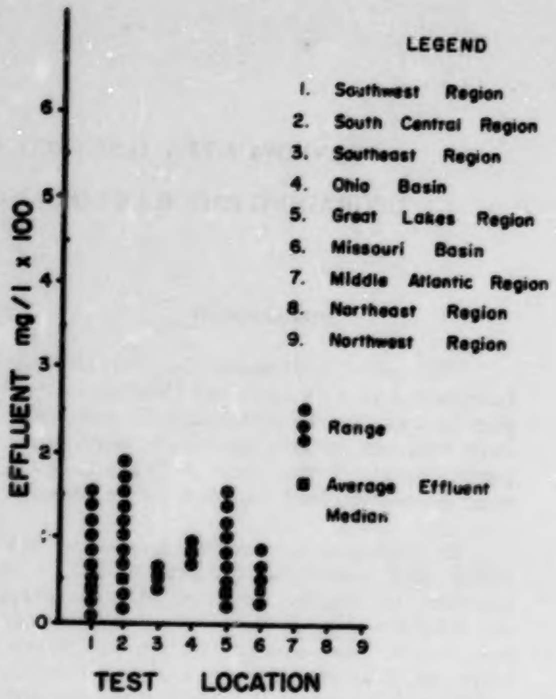

Figure 2. Regional average median effluent values and ranges of values for BOD in facultative lagoons.

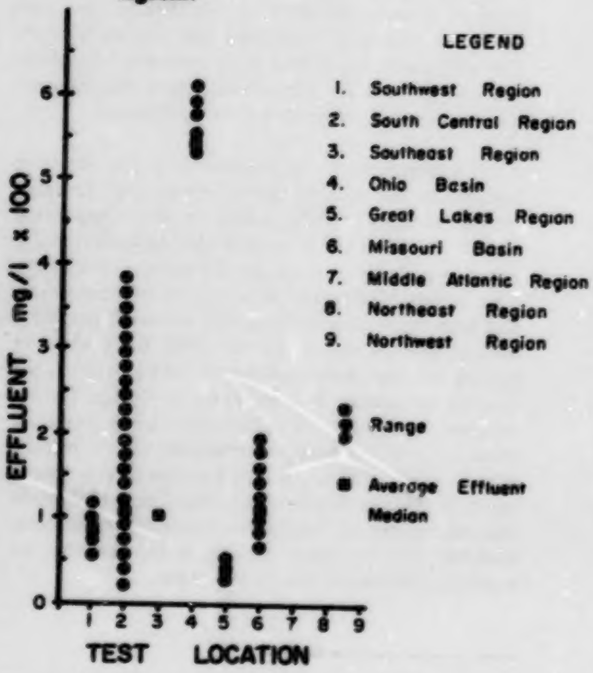

Figure 3. Regional average median effluent values and ranges of values for suspended solids in facultative lagoons.

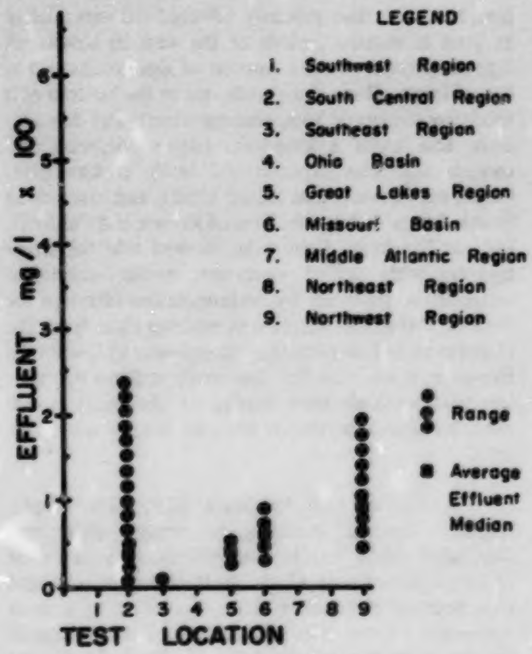

Figure 4. Regional average median effluent values and ranges of values for BOD in aerated lagoons.

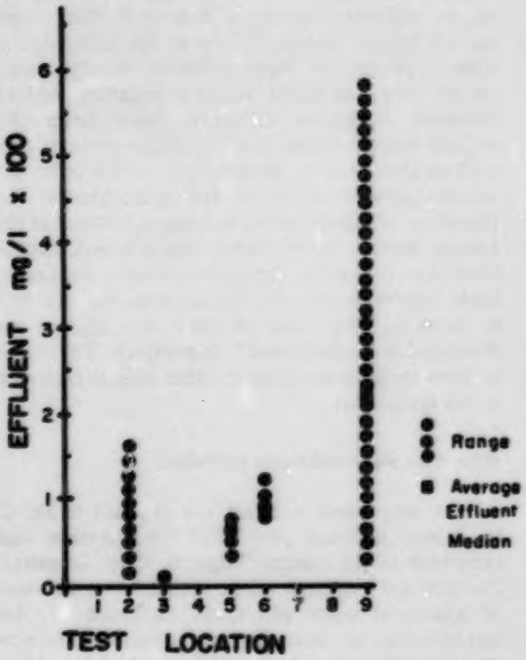

Figure 5. Regional average median effluent values and ranges of values for suspended solids in aerated lagoons.

Previous Lagoon Projects Sponsored by EPA or Predecessor Organizations

Over the period of time covering the increased utilization of lagoons, the EPA and its predecessor organizations have sponsored several meetings and research projects on the general topic of wastewater lagoons and in a few cases on means to upgrade lagoons. These have included the First and Second International Symposium for Waste Treatment Lagoons in 1960 and 1970; the treatment of tanning wastes by an anaerobic-aerobic lagoon system (Parker, 1970); a review manual on "Waste Treatment Lagoons–State-of-the-Art" by McKinney et al. (1971); a project on "Supplementary Aeration of Lagoons in Rigorous Climate Areas" (Champlin, 1971); studies on tertiary treatment of lagoon effluents for phosphorus removal and reuse of the effluent for a recreational lake (Dryden and Stern, 1968); various studies by the EPA staff at the Arctic Research Laboratory in Fairbanks, Alaska, on lagoon treatment in cold climates, including a study on coarse-bubble diffusers for aerated lagoons in cold climates (Christianson, 1973); and the previously mentioned review of "Lagoon Performance and the State of Lagoon Technology" by Barsom (1973). However, this previous involvement at the federal level lacked overall planning and had no organized strategy to systematically encourage improved lagoon design and operation.

Secondary Treatment Standards

The recent promulgation of Secondary Treatment Standards for municipal installations by EPA (Secondary Treatment Information, 1973) has provided the necessary stimulus to develop a vigorous multiple-faceted lagoon upgrading research program. These regulations which will be effective on July 1, 1977, state that effluent BOD (5-day) and suspended solids shall not exceed an arithmetic mean value of 30 mg/l each for effluent samples collected in a period of 30 consecutive days nor shall they exceed 15 percent of the arithmetic mean of the BOD (5-day) and suspended solids values for influent samples collected at approximately the same times during the same period (85 percent removal). For effluent samples collected in a period of 7 consecutive days, the arithmetic mean of the effluent BOD (5-day) and suspended solids values shall not exceed 45 mg/l each. The geometric mean of the value for fecal coliform bacteria for effluent samples shall not exceed 200 per 100 milliliters for samples collected in a period of 30 consecutive days nor 400 per 100 milliliters for samples collected in a period of 7 consecutive days. The effluent values for pH shall remain within the limits of 6.0 to 9.0.

Research Needs Identified for Lagoon Upgrading Program

The major research needs for wastewater lagoons identified over the past several years by EPA regional and headquarters personnel and by a variety of state, academic, and private groups are summarized below:

1. Develop and demonstrate low cost suspended solids and algal removal processes.

2. Establish design guidelines of practice for lagoon effluent discharge to the land.

3. Establish disinfection guidelines and demonstrate applicable methods.

4. Develop and demonstrate nutrient control technology.

5. Characterize the various alternatives of upgrading lagoons so that construction grant funds can best be utilized, and that construction grants personnel will have assurance that a given design will at least have the potential of satisfying applicable effluent standards.

6. Evaluate the utilization of algae to produce a useful produce either as a direct protein supplement for animal feed, or as a source of food for biological predators.

7. Develop and demonstrate methods to control weeds and other undesirable aquatic growths in lagoons.

Fiscal Year 1973 Projects

Removal of algae from lagoon effluents

With the promulgation of the EPA Secondary Treatment Standards and the publication of the Barsom (1973) report, attention was suddenly focused on lagoon treatment technology. It became evident that a careful and comprehensive reappraisal of lagoon capabilities and deficiencies was needed to determine the advisability of new lagoon construction and potential methods for successfully upgrading the more than 4,000 existing lagoons to consistent secondary treatment quality. Methods for solving many of the operational problems such as offensive odors were known by engineering firms and consultants who had kept abreast of the continuing evolution of lagoon designs. This information has been collected and presented at EPA Technology Transfer Design Seminars and published in a Technology Transfer Design Seminar handout. One major problem, however, that seriously affected effluent quality at least in certain periods of the year in almost all lagoon systems was the amount of algae contained in the effluent. These algae settle out in the bottom of a receiving stream or lake, undergo death and degradation, and exert a long-term oxygen demand and oxygen sag. The experimental work at Lancaster, California (Dryden and Stern, 1968), and research in South Africa and by the firm of Brown and Caldwell, Inc., at Stockton, California, showed that for larger lagoons with skilled operators, tertiary chemical coagulation followed by sedimentation-filtration or froth-flotation effectively removed the algae from the effluent as well as removing phosphorus to low levels. However, it was felt that less costly and less operator intensive methods were needed for the smaller communities where personnel are customarily not available.

A Request for Proposals (RFP) for simple, reliable, low-cost methods to remove algae and suspended solids from lagoon effluents elicited a total of 27 responses proposing a great variety of potential algal removal techniques. After evaluation by a panel composed of members of the staff of the Biological Treatment Section of the Advanced Waste Treatment Research Laboratory at the National Environmental Research Center in Cincinnati, three projects were chosen for funding as being most likely to meet the criteria set forth in the RFP. They are passage of lagoon effluent through a slow-rock filter being studied by Dr. Walter O'Brien at the University of Kansas, passage of lagoon-effluent through intermittent slow-sand filters, and crop irrigation and land spreading of lagoon effluents. These latter two projects were combined into a single research contract to Utah State University. This work is under the overall guidance of Dr. E. Joe Middlebrooks. The University of Kansas study is being conducted at the Eudora, Kansas, lagoon (three cells in series), and the Utah State projects are being conducted at the Logan, Utah, lagoon (seven cells, five in series, the first two in parallel). Site plans of these two lagoons are illustrated in Figures 6 and 7, respectively. The details of these projects are given in other reports presented at this symposium.

Manual on algae and water pollution

A sole source contract was awarded to Dr. C. M. Palmer in fiscal year 1973 for a revision and expansion of his manual "Algae in Water Supplies." The title and emphasis will be changed with inclusion of additional color plates, an expanded key for identification of algae, and new chapters. The new chapters will discuss Algae and Eutrophication, Algae and Pollution, Algae as Indicators of Water Quality, Algae in Streams, and Algae in Sewage Stabilization Ponds.

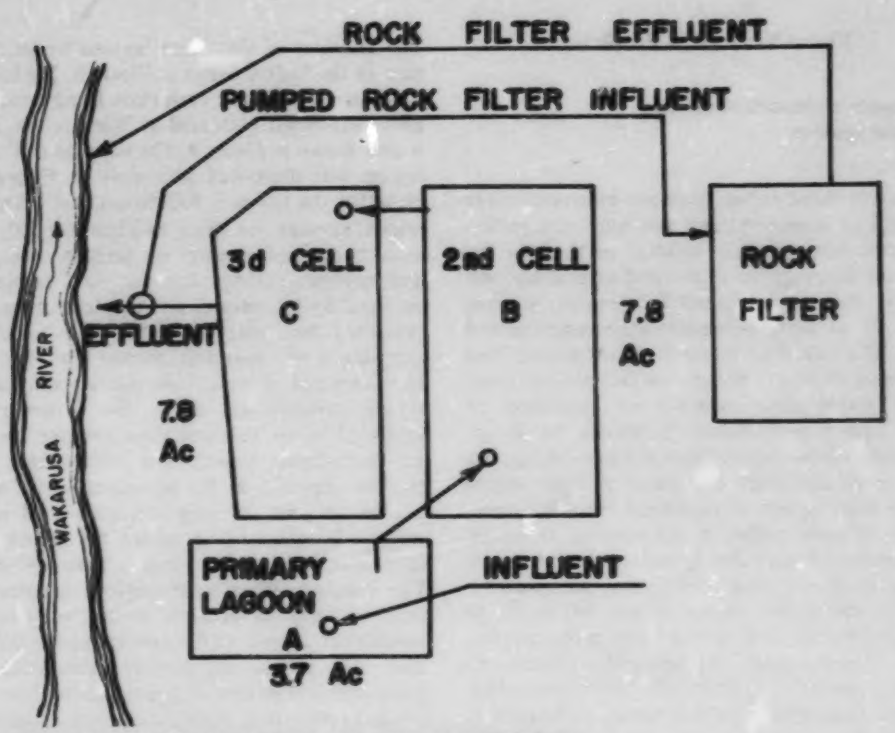
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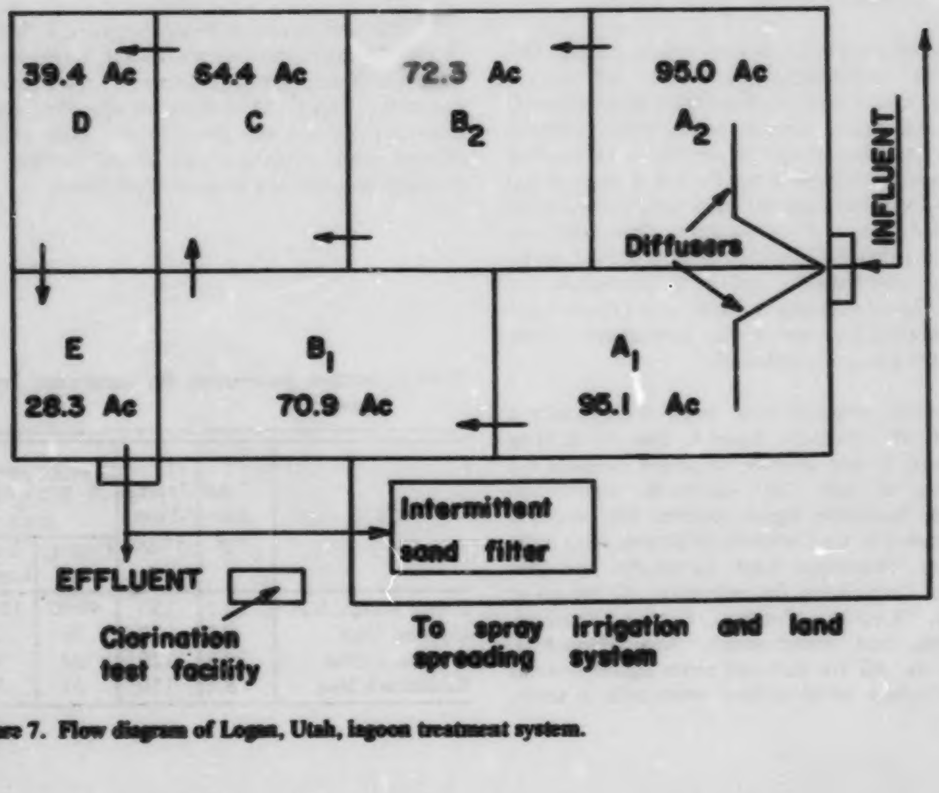
Figure 6. Flow diagram of Eudora, Kansas, lagoon treatment system.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Intermittent
sand filter
EFFLUENT
Clorination
test facility
To spray irrigation and land
spreading system

Figure 7. Flow diagram of Logan, Utah, lagoon treatment system.

Fiscal Year 1974 Projects

Performance evaluation of existing facultative lagoons

As was stated earlier, there is a wide variation in the design of lagoon systems and long-term performance data are generally lacking, particularly for continuous-discharge facultative and aerated lagoons. Typically, there is no formal test program at most lagoons, or at best, infrequent grab sampling and analysis. The EPA Task Force Committee assembled to prepare a Technical Bulletin on Design Criteria for Lagoons found little evidence of evaluation of existing lagoon performance in relation to design. What data do exist indicate that multiple-cell lagoons (series or parallel-series operation) perform better than one large lagoon of equivalent detention time, and that effluent quality is deteriorated either by large amounts of algae during summer periods or by excessive cold and icing over leading to anaerobic conditions during the winter. It was felt to be of utmost importance to determine early in the program whether well-designed, multiple-cell, continuous-discharge lagoons can meet the 1977 Secondary Treatment Standards on a year-round basis without additional treatment. To accomplish this, it was decided that a performance evaluation of several lagoons in different climates and geographical locations of the country was an essential first step.

Qualified contractors were solicited via an RFP to submit well-designed facultative and aerated candidate lagoon systems amenable to rigorous sampling and analytical evaluation for a period of one year. Twenty-four hour composite sampling was specified on a twice-a-week basis except for four periods during the year (once each season) when sampling was to be conducted for 30 consecutive days. It was emphasized that the candidate lagoons should not be presently over-loaded, and that provisions for accurate flow measuring (influent and effluent) had to be assured. Lagoons serving populations greater than 5,000 were not considered.

Sixteen proposals were received comprising a total of 39 candidate lagoons. Due to funding constraints, it was possible to award contacts for evaluation of only four multi-cell, continuous-discharge, facultative lagoon systems. The contracts were awarded to the University of Kansas, Utah State University, Mississippi State University, and JBF Scientific Corporation for evaluation of lagoons at Eudora, Kansas; Corinne, Utah; Kilmichael, Mississippi; and Peterborough, New Hampshire, respectively. All are three-cell series lagoon systems except Corinne which utilizes seven cells in series. The locations of these four lagoons are spotted on a map of the United States in Figure 8. The location of a similar performance evaluation being conducted by EPA Region VII personnel at Blue Springs, Missouri, is also shown in Figure 8. The site plan of the Eudora lagoon was illustrated previously in Figure 6. Site plans for the Corinne, Kilmichael, and Peterborough lagoon systems are given in Figures 9, 10, and 11, respectively. Information on acreage, average flow, and organic loadings for the four lagoons being surveyed by the research and development program is presented in Table 1. Table 2 summarizes the sampling and analytical format for the project. Measurements of daily flow entering and leaving the lagoon systems will allow the determination of long-term water balances (accounts for evaporation and percolation losses). The enumeration of fecal coliform bacteria in the lagoon effluents will show whether or not the long detention time in lagoon systems is sufficient to reduce the numbers to the levels specified in the secondary treatment standards. The Peterborough system employs chlorination before discharge of effluent to the receiving water. Soluble BOD and COD measurements before and after chlorination will provide information on the possible solubilization of organic matter due to lysing of algal cells arising from the application of chlorine.

Lagoon Workshop Symposium

This symposium is being sponsored with fiscal year 1974 funds and was planned to accelerate the exchange of information between the research staffs conducting experimental work on upgrading lagoon treatment systems and the EPA and state officials charged with regulatory and permit activity and decisions on construction grant applications.

Table 1. System parameters for facultative lagoon survey.

Site	Total Acreage	Average Flow (mgd)	Lagoon Loadings (lb BOD_5/day/acre)	
			Primary Cell	Total Lagoon
Peterborough, N.H.	20.7	.150±	40-45	15-20
Corinne, Utah	9.5	.070	35	14
Eudora, Kansas	19.3	.120	34	13
Kilmichael, Miss.	8.1	.110±	51	33

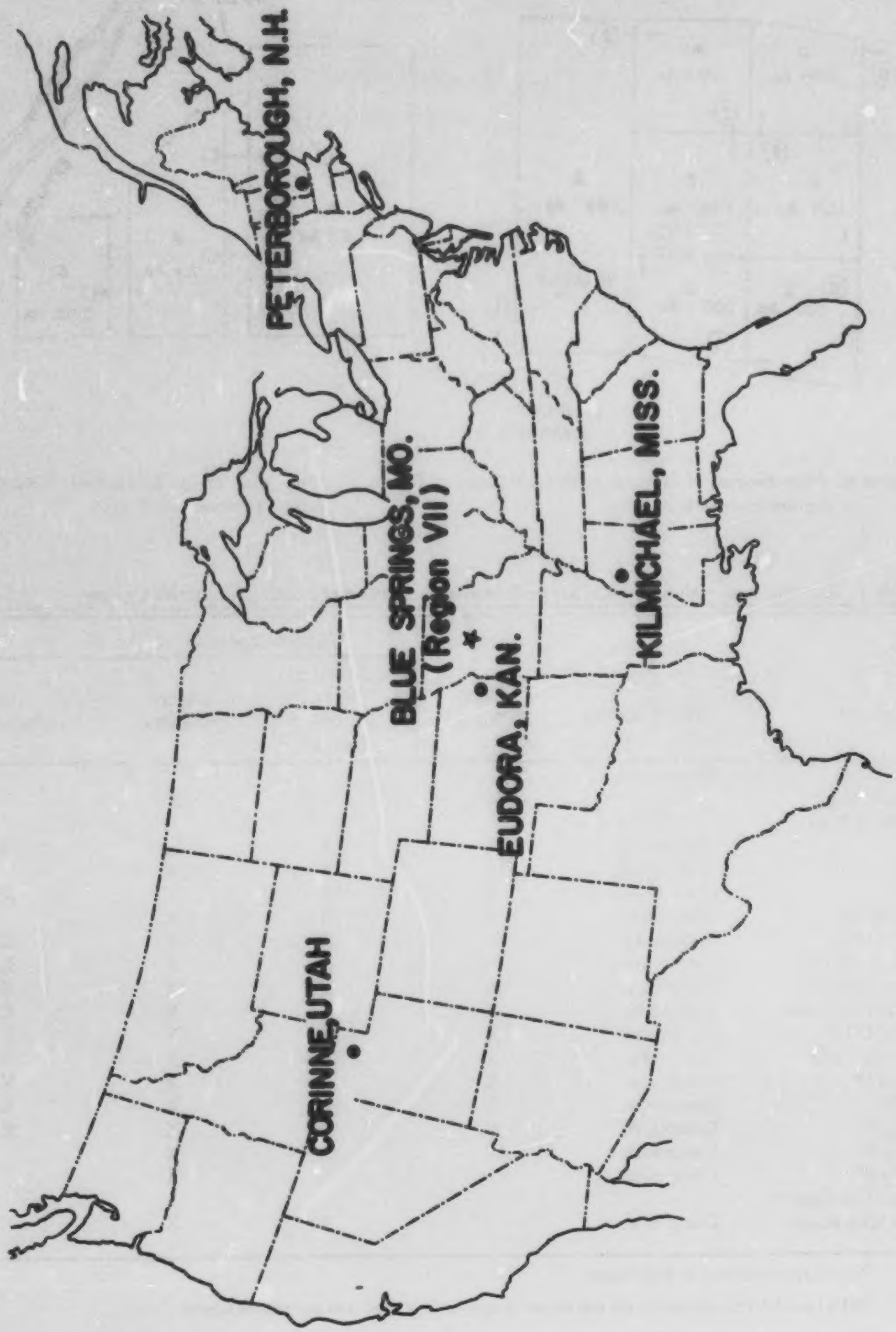

CORINNE, UTAH
EUDORA, KAN.
BLUE SPRINGS, MO.
(Region VII)
KILMICHAEL, MISS.
PETERBOROUGH, N.H.

Figure 8. Location of facultative lagoon treatment systems being tested for year-round performance.

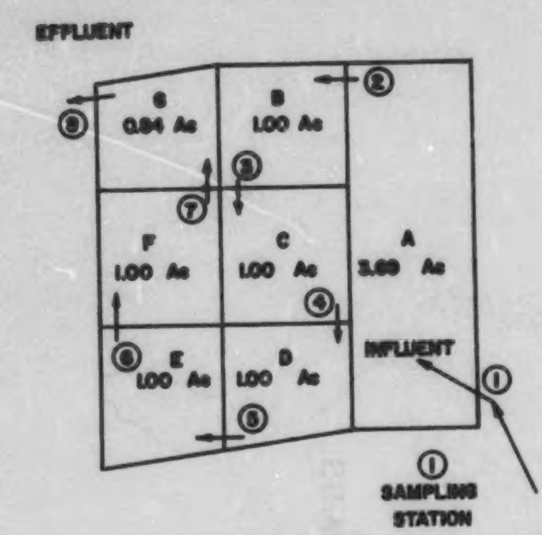

EFFLUENT
⑧
G
0.84 Ac
②
B
1.00 Ac
③
⑦
F
1.00 Ac
C
1.00 Ac
A
3.69 Ac
④
⑥
E
1.00 Ac
D
1.00 Ac
INFLUENT
①
⑤
①
SAMPLING
STATION

Figure 9. Flow diagram of Corinne, Utah wastewater lagoon treatment system.

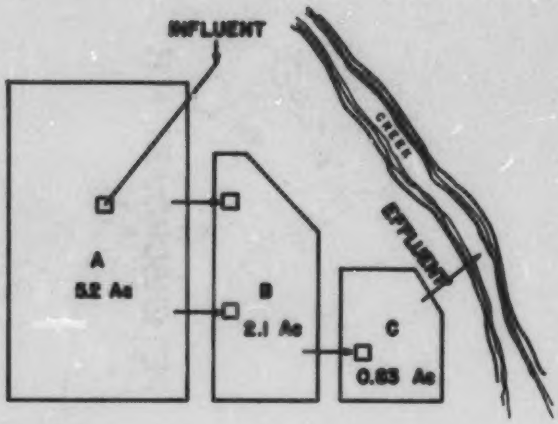

INFLUENT
CREEK
A
5.2 Ac
EFFLUENT
B
2.1 Ac
C
0.83 Ac

Figure 10. Flow diagram of Kilmichael, Mississippi, lagoon treatment system.

Table 2. Sampling and analytical guide for performance evaluation of existing facultative lagoons.

Parameter	Type of Sample	Lagoon Influent	Sample Location: Between Cells of Multi-Cell Lagoons	Lagoon Effluent	After Chlorination
WW Flow	Continuous				
Daily Total		X		X	
Min. & Peak		X			
pH	Grab, in-situ	X	X	X	X
WW Temp.	Grab, in-situ	X	X	X	
D.O.	Grab, in-situ	X	X	X	X
Alkalinity	Composite	X	X	X	
Total BOD_5[a]	Composite	X	X	X	X
Soluble BOD_5[a]	Composite	X	X	X	X
Susp. Solids[b]	Composite	X	X	X	X
Fecal Coliforms	Composite	X	X	X	X
Total COD[c]	Composite	X	X	X	X
Soluble COD[c]	Composite	X	X	X	X
Total P[c]	Composite	X		X	X
TKN[c]	Composite	X		X	X
NH_3-N[c]	Composite	X		X	X
NO_2-N[c]	Composite			X	
NO_3-N[c]	Composite			X	
Algae Cell Count by Microscope	Composite		X	X	X

[a] Nitrification inhibited in BOD bottle.

[b] MLSS and MLVSS analysis (grab) will be performed on the contents of the aerated lagoon.

[c] Analyses to be performed in Cincinnati.

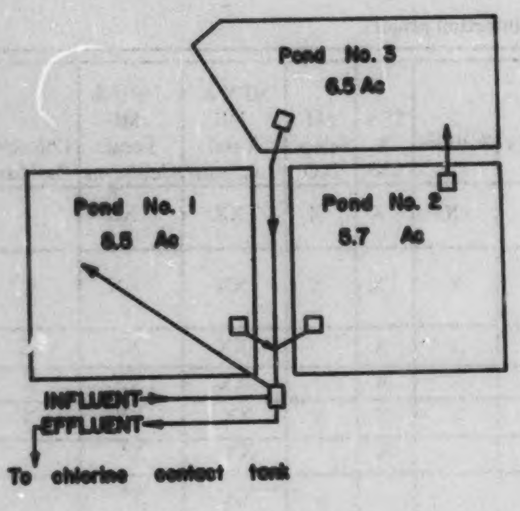
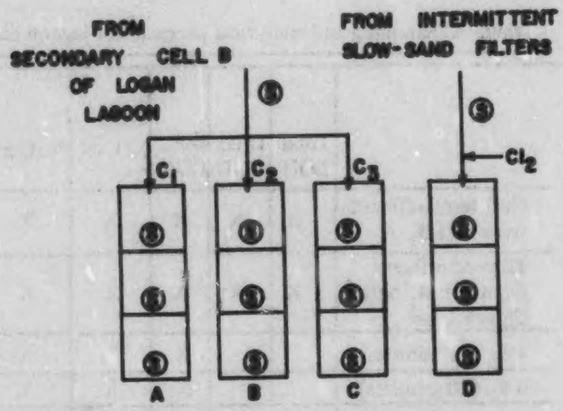


Figure 11. Flow diagram of Peterborough, New Hampshire, lagoon treatment system.



1. C_1, C_2, C_3, represent different Cl_2 conc.
2. Ⓢ denotes sample locations for chlorine contact tank influents & at detention times of 15, 30, & 45 min.

Figure 12. Process test sequence for lagoon chlorination study.

Fiscal Year 1975 Projects

Disinfection study (planned)

A scope-of-work has been planned and a contract is being prepared for a chlorine disinfection study to be carried out by Utah State University at the Logan, Utah, lagoon. The site plan for the Logan lagoon system was shown previously in Figure 7. Effluent to be disinfected can be taken from the intermittent slow-sand filters, from cell B_1 (secondary cell), or from cell E (final effluent). This will enable evaluation of a variety of concentrations of fecal coliform bacteria, algae, and suspended solids in the effluents to be disinfected. Four experimental chlorine contact tanks will be constructed with baffles to control short circuiting. The flow of effluents through each tank will be precisely controlled. Samples will be analyzed 2 days per week on tank influents and at the 15-, 30-, and 45-minute detention points. Three of the chlorination systems will be operated in parallel with different chlorine concentrations using effluent from either cell B_1 or cell E as tank feed. The other experimental system will evaluate chlorination of effluent from the intermittent slow-sand filters. A diagram of the proposed chlorination experimental layout is presented in Figure 12. A summary of the sampling and analytical program for this project is given in Table 3. The objectives of this comprehensive disinfection project are to learn more precisely the factors affecting disinfection of lagoon effluents and the conditions under which chlorine dosing will release soluble organics through lysis of algal cells. As can be seen in the sampling schedule, the die-off of fecal coliforms from cell to cell of the Logan lagoon will also be studied.

Nitrification of lagoon effluent with a rotating biological contactor (planned)

Nitrification does not normally occur in facultative lagoon systems because the concentration of nitrifying bacteria in the upper aerobic zone is not sufficient for significant oxidation of ammonia nitrogen. The numbers of nitrifying bacteria may be low because of inhibition by the algae or because the nitrifying bacteria tend to adsorb onto soil particles and other aggregates and settle out into the anaerobic zone where they cannot carry out their activities. Many lagoon treatment systems, thus, may have high levels of effluent ammonia nitrogen (20 to 30 mg NH_4^+-N/l) that can seriously interfere with chlorine disinfection, can exert an oxygen demand in the receiving waters if nitrification occurs in those waters, and may be a major cause of eutrophication in receiving waters.

A scope-of-work has been discussed with the University of Michigan to conduct a lagoon nitrifica-

Table 3. Sampling and analytical program for lagoon disinfection project.

	Total BOD	Total COD	Sol. COD	NH_3-N	Turbidity	Sulfides	TSS & VSS	pH Temp. D.O.	MPN & MF Total Coliforms	MPN & MF Fecal Coliforms	Chlorine Residual
Unfiltered effluent from Cell B_1	X	X	X	X	X	X	X	X	XX	XX	
Filtered effluent from Cell B_1 before Chlorination	X	X	X	X	X	X	X	X	XX	XX	
4 θ_1 (15 minutes)			X	X	X	X	X		XX	XX	X
4 θ_2 (30 minutes)			X	X	X	X	X		XX	XX	X
4 θ_3 (45 minutes)		X	X	X	X	X	X	X	XX	XX	X
Raw Wastewater	X	X	X	X			X		XX	X[a]	
Effluent from Cells A_1 & A_2								X	XX	X[a]	
Effluent from Cells B_2 & E								X	XX	X[a]	
Effluent from Cells C & D								X	XX	X[a]	
Analyses/day of sampling	3	7	15	15	14	14	15	12	42	35	12

[a] MPN not required.
θ = mean residence time.

tion grant project at the Genoa, Ohio, wastewater lagoon system. This is a two-cell facultative lagoon as shown in the site plan in Figure 13. The project will investigate if it is possible to nitrify the effluent from a continuous-discharge lagoon using a rotating biological contactor (RBC) unit (also known as rotating biological discs or rotating biological surface), and will attempt to quantify seasonal effects. A six-stage RBC pilot plant will be utilized with 15 4-foot diameter discs in each stage yielding a total of 90 discs. This provides 2,272 sq ft of disc surface area and with a nominal loading of 2.6 gpd/sq ft will require a throughput of 5,900 gpd. Pumpage will be variable over a range down to 0.5 gpd/sq ft. Composite samples of the RBC feed (lagoon effluent) and after each of the six stages will be collected three times per week. Analyses to be performed include organic, ammonia, nitrite and nitrate nitrogen, BOD, COD, suspended solids, biomass on the discs, pH, wastewater temperature, and alkalinity. Adjustments of the flow and organic and ammonia loadings will be made to optimize the biological response. The effects of seasonal temperatures on the kinetics of nitrification will be observed and the best configuration of the six stages (series, parallel, or a combination thereof) defined. The work plan also calls for loading the system to failure of nitrification and then noting

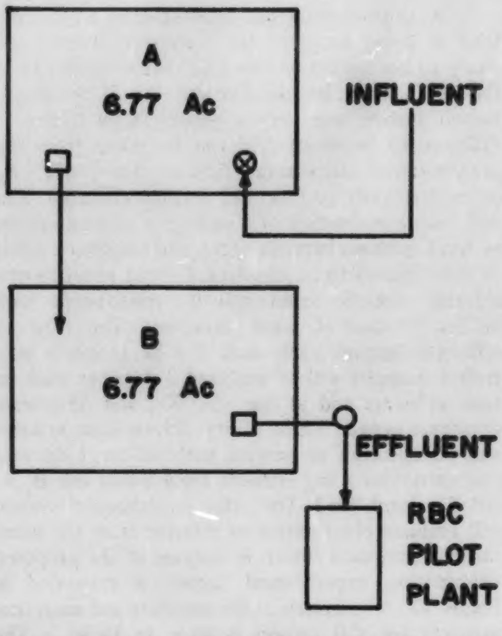


Figure 13. Flow diagram of Genoa, Ohio, lagoon treatment system.

the rate of recovery of nitrification upon the resumption of normal loading.

Hopefully, this project will generate design criteria for future full-scale work and will eventually prove the RBC approach to be a simple, reliable method for nitrification of lagoon effluents.

Combined phosphorus and nitrogen control in a lagoon (planned)

A joint inhouse/extramural project is being planned to begin early in calendar year 1975 to remove phosphorus from and nitrify the effluent of the St. Charles, Maryland, lagoon. The site plan shown in Figure 14 indicates this is a six-cell series lagoon with the first two cells being aerated followed by four facultative cells. The average flow to this six-cell system is 0.6 mgd. Equipment installation and overall project direction will be the responsibility of the staff of the Joint EPA/District of Columbia Pilot Plant located several miles away. A service contract with the Charles County (Maryland) Community College is being contemplated to provide necessary day-to-day operation, weekend surveillance, and analytical work.

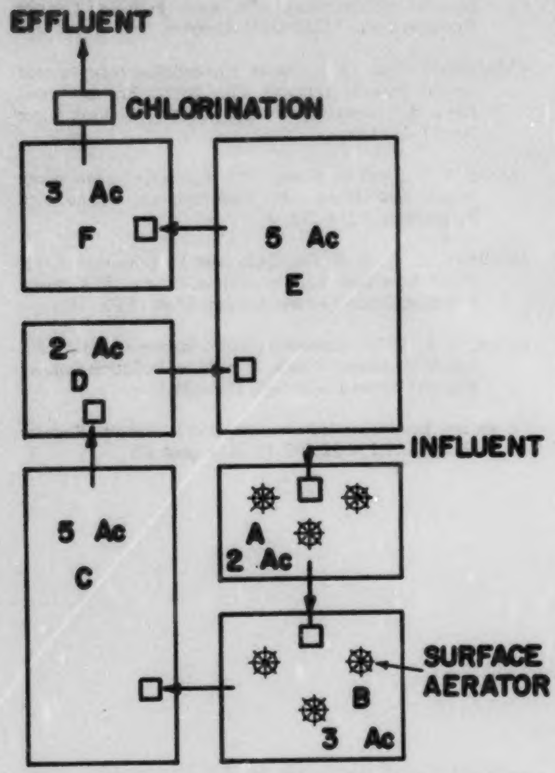


Figure 14. Flow diagram of St. Charles lagoon treatment system–site of planned nutrient control project.

Phosphorus removal will be effected by the addition of alum to different cells of the lagoon system. Studies will be conducted to determine which of the first two or three cells is optimal for phosphorus removal. The rate of sludge buildup at the bottom of the cells will be followed. The chemical feed system will be designed to provide good mixing with cell influent wastewater. A most important observation will be whether the precipitated phosphorus sludge tends to settle out in a "clump" or evenly distributes itself across the cell bottom. This will have ramifications concerning the frequency at which the chemical sludge will have to be removed by "clamming" and also the possible resuspension of precipitation phosphorus particles by wind action.

The nitrification studies on the lagoon effluent will be similar to those planned at Genoa, Ohio, with the exception that a plastic-media trickling filter pilot unit will be used instead of an RBC pilot system. The purpose of the study and parameters to be evaluated will nearly parallel that work.

This project should aid in the development of guidelines for design and operation to simultaneously remove phosphorus and oxidize ammonia nitrogen for those lagoons faced with both eutrophication and excessive oxygen depletion in their receiving waters.

Summary of Lagoon Upgrading Projects

A summary of fiscal year 1973, 1974, and 1975 research and development sponsored lagoon upgrading projects and the cost of each project are given in Tables 4 and 5, respectively. For fiscal year 1975, lagoon upgrading received the largest share of research and development funds of any of the biological wastewater treatment processes.

Table 4. List of EPA lagoon projects FY '73-75.

1. Algae Removal (FY '73)
   - A. Intermittent slow-sand filter, Utah State University
   - B. Crop irrigation and land spreading, Utah State University
   - C. Slow-rock filter, University of Kansas
2. Manual on Algae and Pollution (FY '73), Dr. Palmer
3. Facultative Lagoon Performance Survey (FY'74)
4. Lagoon Symposium (FY '74)
5. RBC Nitrification Following Lagoon (FY'75 Planned)
6. Phosphorus Precipitation in and Plastic Media Nitrification Following Lagoon (FY'75 Planned)
7. Lagoon Disinfection (FY'75 Planned)
8. Aerated Lagoon Performance Survey (FY'75 Tentative)

Table 5. Summary of Ord funding for lagoon upgrading projects.

Description	Amount ($ K)		
	FY'73	FY'74	FY'75
1. Manual on Algae in Water Pollution	12		
2. Algae Removal Projects	322		
3. Facultative Lagoon Performance Survey		148	70
4. Lagoon Symposium		10	
5. RBC Nitrification Following Lagoon (Planned)			49
6. Phosphorus Removal in and Plastic Media Nitrification Following Lagoon (Planned)			166
7. Lagoon Disinfection Project (Planned)			105
8. Aerated Lagoon Performance Survey (Tentative Based on Availability of Funds)			(350)
TOTALS	334	158	390 (350)

Future Planned Studies

Sufficient funds were not available in fiscal year 1974, nor are they as yet available in fiscal year 1975, to include well-designed aerated lagoons and aerated/facultative lagoon combinations in the performance survey currently beginning on existing facultative lagoons only. It is hoped that funds will yet be made available in fiscal year 1975 for this important survey. If not, the project will be programmed into the fiscal year 1976 budget. It is anticipated that $350,000 - 400,000 will be required to evaluate four to five such lagoons for 1-year each.

In future years, it is hoped that studies can be undertaken to evaluate the use of algae as food sources for animals or predators. It is probable that unique designs to limit the predators to certain cells of a lagoon system will be needed and that the factors affecting the development of the predators will have to be understood to develop a successful scheme of operation. Additionally, to create the necessary credibility and confidence to convince design engineers to utilize the upgrading techniques currently being researched, full-scale demonstrations of the more successful methods of removing algae from the controlling nutrients in lagoon effluents are planned. The principal objective of the entire lagoon upgrading program is to develop design and operating guidelines to meet a variety of effluent quality needs, and to do so with an array of upgrading techniques applicable to the small community nature of lagoon users. Only in this way can the relative simplicity of operation and maintenance and the low cost that have made lagoons such a popular method of wastewater treatment for small communities in the past be retained.

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STATE OF THE ART OF LAGOON WASTEWATER TREATMENT

R. E. McKinney[1]

Oxidation ponds have had extensive use in the United States for the treatment of domestic and industrial wastewaters from small communities and isolated industrial plants. The advantages of the oxidation pond systems lie in their simplicity of design, construction, and operation. No other wastewater treatment system is as easy to design, construct, or operate. In spite of its simplicity, the oxidation pond system produces a high degree of wastewater stabilization. In recent years with greater emphasis on wastewater treatment, some concern has been raised as to effluent quality produced by oxidation ponds. With the EPA requirements for secondary treatment clearly requiring a 30 mg/l BOD_5 and a 30 mg/l suspended solids for all municipal effluents, the oxidation pond has been signaled out for its potential failure to meet these standards. Unfortunately, few people responsible for enforcement of EPA regulations are concerned about wastewater treatment and the consequences of eliminating oxidation ponds as wastewater treatment systems. While oxidation ponds have been in use for many years, there have been few documented cases of serious pollution problems created by the effluent discharged from oxidation ponds. The reason for this is that oxidation ponds are essentially small treatment devices that produce a relatively small discharge. The nature of the BOD and the suspended solids from oxidation ponds are different than the discharges from other treatment devices; but this difference has largely gone unnoticed. It is time that we recognize that the BOD_5 and the suspended solids discharged from an oxidation ponds system are definitely different than the BOD_5 and the suspended solids from other treatment systems. The impact on the receiving waters is different even though they are measured in the same units. Unfortunately, there has been a tendency to lump everything together into a single simple standard that can be easily administered without any thinking or understanding of either the problem or its solution.

Current Design Concepts

The design criteria for oxidation ponds developed empirically by trial and error. Engineers took the data from California and Texas and applied it to other locations throughout the country. They soon learned that California and Texas were different from their part of the country and that oxidation ponds worked differently in different parts of the country. More was learned from the initial failures than from the initial successes. Oxidation ponds were built too small as well as too large. Both systems produced problems which required further engineering before solutions were developed. Neither system produced impossible solutions. As information was developed through field experience, it was possible to evolve a sound set of design criteria that fit different parts of the country.

Unfortunately, little operational data were generated from oxidation ponds. Oxidation ponds were too small to require extensive analytical data. Simplicity of operation eliminated the need for operators and little data were generated. A few tests were made periodically that showed that oxidation ponds performed reasonably well but the data were limited. When problems occurred, more extensive sampling was made to confirm that a problem existed and to base a case for requiring expanded treatment. Slowly, a large mass of data began to be accumulated on problem ponds and a question was raised as to whether or not oxidation ponds should be used for wastewater treatment. The question is a valid one which must be answered in both technical and nontechnical terms. There is a need to know exactly how oxidation ponds function and exactly what kind of effluent can be produced on a day in and day out basis. There is also a need to examine the alternatives to oxidation ponds and their impact on society if it is necessary to replace oxidation ponds.

Oxidation ponds are simply large flat holes in the ground with adequate capacity to hold the wastewaters for several months. It was found that oxidation ponds should not be too shallow as weeds would grow up and create a place for mosquitoes to grow. At the same time the ponds should not be too deep as there has to be adequate surface for algae to grow and to produce sufficient oxygen to keep the system aerobic at all times. Experience indicated that a 4-foot water depth was reasonable for all parts of the country except the desert southwest where evaporation required deeper ponds. It was soon learned that flexibility to control the water depth in the ponds was also desirable. Often effluent control structures permitted operation between 3 and 5 feet. The ponds were drawn down to minimum depth once cold weather set in and were allowed to rise up to maximum depth by spring time. In this way the sewage was actually retained during the low temperature periods and discharged when maximum flow conditions existed in the receiving stream.

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The size of oxidation ponds was dictated by the population served with a theoretical volume of wastewater per capita. In the southwestern part of the country the rate of evaporation during the summer dictated the size of the ponds while the length of winter dictated the size of ponds in northern climates. For the most part, the initial ponds were conservatively designed with 120-180 day retention periods. As results indicated satisfactory operation, engineers slowly reduced the size of the ponds down to 90 days and then to 60 days in some instances. With the reduction in retention time came an increase in organic load. At 180 days retention the theoretical design population was 72 people/acre with a BOD_5 load of 12 lbs/acre/day. The more convenient design parameter soon became population per acre since the calculation was made so easily. At 100 people/acre the theoretical retention time became over 120 days and the load was 17 lbs BOD_5/acre/day. At 200 people/acre the retention period was reduced in half and the organic load doubled. As engineers became more sophisticated, the population/acre design criteria was replaced by the BOD_5/acre/day loading rate. A 40 lb BOD_5/acre/day loading rate became popular as the basis of design of oxidation ponds. Even a 65 lb BOD_5/acre/day loading rate was employed; but problems began to occur and the loading rate was reduced. At 65 lbs BOD_5/acre/day there was an increase in operational problems and engineers have tended to back off from such loads even though the literature indicates that operation as high as 100 lbs BOD_5/acre/day should be possible without problems. For the most part engineers have relied more on field evaluations than on laboratory scale or even pilot scale research studies over a relatively short period of time. It should be recognized that no one likes to talk about treatment systems that fail to operate satisfactorily. For this reason it has been extremely difficult to obtain data necessary to develop sound design criteria. Each state regulatory agency seems to have arbitrarily developed a set of criteria that they felt comfortable with and have proceeded to use. For the most part there is little scientific or technical basis for the design criteria.

Biological Concepts

In order to properly design oxidation ponds it is essential to understand the biology and the biochemistry occurring within the ponds. Over the years considerable information has been developed on various aspects of oxidation ponds but seldom has the information been put together in a usable format for the design engineer. The net effect has been that the engineer has simply followed the state health department design criteria without any regard to the results obtained. Too often, if problems resulted, the engineer was quick to point out that the system followed state design criteria and the problem was not his responsibility.

Initially, the biochemical reactions were thought to be a simple set of symbiotic reactions between the bacteria and the algae. The bacteria aerobically stabilized the organic matter in the wastewaters to carbon dioxide and converted it back to oxygen which the bacteria needed for their aerobic reaction. All that was needed was time for the bacteria to metabolize the organic matter and adequate surface for the algae to obtain enough light for their metabolic reactions. The simple symbiotic reaction was not as [illegible] it seemed at first. The bacterial reacti[illegible] of synthesis, followed by endogenous [illegible]. The synthesis reaction requires oxygen [illegible] the metabolism of the organic matter to form new cell mass, carbon dioxide, water, and ammonia. The following reaction is representative of the metabolism of domestic sewage by bacteria in a mixed culture environment. Only the biodegradable fraction of the domestic sewage is used in this equation.

Synthesis:

$$C_{7.1}H_{12.6}O_{3.0}N + 3.0\,O_2 \rightarrow C_5H_9O_3N + 2.1\,CO_2 + 1.8\,H_2O$$

With a normal domestic sewage having 200 mg/l BOD_5, 112 mg/l oxygen would be required to stabilize the organic matter in the sewage and 154 mg/l VSS as microbial solids would be created. These microbial solids are unstable organic matter and continue to undergo aerobic degradation via endogenous respiration as follows:

Endogenous Respiration:

$$C_5H_9O_3N + 4\,O_2 \rightarrow 0.2\,C_5H_9O_3N = 4\,CO_2 + 0.8\,NH_3 + 2.4\,H_2O$$

The endogenous respiration reaction results in the production of 30 mg/l inert organic solids as dead microbial cells while using 150 mg/l oxygen and releasing 13 mg/l ammonia nitrogen and 206 mg/l carbon dioxide. The inert organic solids are still suspended solids that settle very slowly since they were individual bacteria cells. The empirical formulation used in the above equations was $C_5H_9O_3N$ instead of the more classical $C_5H_7O_2N$. The reason for this is that a number of studies over the past 20 years have indicated that $C_5H_9O_3N$ is the more correct value. The most recent study was made at the University of Kansas this past year and produced extensive data on the validity of the $C_5H_9O_3N$ value.

These equations show the two basic bacterial metabolism reactions at completion but give no information about the degree of completion and the time required to reach the completion of each phase. An oxidation pond system is not a concentrated biological reactor like a trickling filter or activated sludge, but rather is a dispersed biological reactor like the BOD bottle. Under aerobic conditions the bacteria will complete the synthesis reaction in 24 to 48 hours and the endogenous respiration reaction in an additional 18 days. From a practical point of view over 99+ percent of the metabolizable organic matter in the domestic sewage will have been metabolized at the end of the synthesis phase. If the bacteria were not dispersed, they could be quickly separated with the production of a high quality effluent. In most oxidation ponds the bacteria metabolism will be complete with the production of 30 mg/l VSS and about 15 mg/l inorganic SS, 314 mg/l carbon dioxide, and 13 mg/l ammonia nitrogen while demanding 262 mg/l oxygen.

The metabolism of the organic matter dispersed in the domestic sewage results in a relatively clear liquid containing considerable carbon dioxide, ammonia nitrogen, and phosphates. In the presence of light energy the algae are able to convert these elements into new cells by a synthesis reaction similar to the bacterial synthesis reaction.

Synthesis:

$$5\,CO_2 + NH_3 + 3\,H_2O \xrightarrow[\text{Algae, PO}_4]{\text{Sunlight}} C_5H_9O_3N + 5\,O_2$$

Unfortunately, the algae reverse the stabilization reaction and take stable materials and convert them back to unstable organic compounds in the form of algal protoplasm. If the 314 mg/l carbon dioxide produced by the bacterial metabolism reactions were all metabolized by the algae, 20 mg/l ammonia nitrogen would be required to synthesize 187 mg/l VSS as algal cells and 229 mg/l oxygen would be released. If the reactions stopped there, it would be a standoff with all the organic matter being converted from domestic wastewater organics to bacteria and algal cell mass. The oxygen demand by the bacteria would exceed the oxygen production by the algae, 262 mg/l oxygen demand in contrast to 229 mg/l oxygen production. If an additional source of oxygen were not readily available, the oxidation pond could not remain aerobic. There is also another factor that has to be considered. The algae cells are unstable organic matter the same as bacteria cells and also undergo endogenous respiration.

Endogenous Respiration:

$$C_5H_9O_3N + 4\,O_2 \rightarrow 0.2\,C_5H_9O_3N + 4\,CO_2 + 0.8\,NH_3 + H_2O$$

The endogenous respiration reaction of the algae is exactly the same as the endogenous respiration reaction of the bacteria. Approximately 37 mg/l of dead algal cells will remain as inert VSS with 19 mg/l inorganic suspended solids. The oxygen demand would be 183 mg/l with 16 mg/l ammonia nitrogen released. The overall reaction would result in a combined suspended solids production of 67 mg/l VSS and an oxygen deficit of 216 mg/l from a wastewater having an initial BOD_5 of 200 mg/l. If the reactions stopped here there would be a serious problem. Fortunately the reactions continue.

Algae undergo endogenous respiration like bacteria; but with the release of carbon dioxide, new algal cells are produced as long as there is sunlight available for metabolism. The net effect is that the algae recycle their carbon from active protoplasm to gas back to active protoplasm with a small fragment, approximately 20 percent, being shunted off to inert volatile organic suspended solids. The two reactions occur simultaneously and pass almost unnoticed. During the dark period algae demand oxygen the same as bacteria and synthesis must wait until light is available.

The cycling between synthesis-endogenous respiration and back to synthesis is not enough to keep the system aerobic nor will it solve the balance between the bacteria and the algae. Something more is needed. That something more is the alkalinity that exists in the sewage. The alkalinity forms a source of carbon for the algae to grow, as shown below.

Alkalinity:

$$10\,NaHCO_3 + NH_3 \xrightarrow[\text{Algae, PO}_4]{\text{Sunlight}} C_5H_9O_3N + 5\,Na_2CO_3 + 5\,O_2 + 2\,H_2O$$

The production of oxygen requires the conversion of inorganic carbon to organic carbon. If 300 mg/l alkalinity as $CaCO_3$ were present in the domestic wastewaters, the algae would convert half of it to the carbonate form and raise the pH while producing 79 mg/l VSS and 96 mg/l oxygen. Unfortunately the additional oxygen is produced at the expense of additional organic matter. The combination of organic metabolism, alkalinity, and time permits the oxidation pond to produce the oxygen needed for waste stabilization. For the oxidation pond to have a net gain in oxygen, it is necessary for the algae to recycle the carbon through several cycles to create inert organic solids that are not oxidized in the oxidation pond. With domestic sewage as used in the previous equations, the minimum balanced retention period would be 160 days. Fortunately, sedimentation of both organic solids in the raw wastes and algae has resulted in balanced operations at 90 to 100 days retention instead of 160 days.

The key to successful production of quality effluents from oxidation ponds lies in algal removal. Most of the efforts to reduce the algae in the effluents from oxidation ponds have centered on effluent structures drawing off the effluent from below the pond surface. Since light is critical for the algae, the motile algae tend to predominate over the nonmotile forms in oxidation ponds and migrate to the desired level near the pond surface. The predominant nonmotile algae tend to have a high surface area to mass ratio so that they easily remain dispersed when agitated slightly. Wind action over the surface of oxidation ponds tends to keep the nonmotile algae suspended while the motile algae have no difficulty remaining suspended. The nonmotile algae settle slowly and eventually are removed by sedimentation where wind action is minimal. The use of multiple series ponds results in decreasing algal concentrations where submerged pipes are used between ponds. In effect, the algae retention time is increased over the liquid retention time. Unfortunately, the effluent quality from multiple cell oxidation pond systems is not satisfactory unless the retention period is quite long, 120-180 days, or unless mixing action is minimized. Recent studies on algal removal from oxidation pond effluents center on rock filtration and sand filtration. It is hoped that information will be forthcoming to demonstrate the validity of these techniques.

Coliform Reductions

In addition to BOD and SS reductions, EPA regulations require coliform reductions to 200 fecal coliforms per 100 ml. The reduction in fecal coliforms follows normal die-off relationships. There are no antibiotics or toxic materials produced in the oxidation ponds that reduce the fecal coliforms. The only factors affecting fecal coliforms are starvation and predation. The lack of sufficient organic matter results in starvation of the fecal coliforms while certain protozoa and crustaceans act as predators on the fecal coliforms the same as on the other bacteria in oxidation ponds. Overall, it appears that multicell ponds will be needed with a total retention period between 60 and 100 days to meet EPA effluent requirements without chlorination. Mixing by wind action in single cell ponds will permit short circuiting and make it all but impossible to meet effluent criteria without chlorination.

Microbiology of Oxidation Ponds

While an understanding of microbiology is not essential to the design of oxidation ponds it does help understand the reactions which occur and the changes that take place during the different seasons of the year. The primary bacteria in oxidation ponds appear to be *Achromobacter*, *Pseudomonas*, and *Flavobacterium* and coliforms. All of the bacteria grow quickly until the organic matter is stabilized and then die off as a result of simple starvation.

The algae which grow in oxidation ponds are related to the chemical quality of the system. For the most part the motile flagellated algae predominate since they can easily move to the optimum light level. *Chlamydomonas*, *Euglena*, *Chlorogonium*, *Phacus*, *Pandorina*, and *Carteria* are the major green flagellated algae predominating in oxidation ponds. The predominant green nonflagellated algae include *Chlorella*, *Ankistrodesmus* *Microactinium*, *Scenedesmus*, *Actinastrum*, and *Closterium*. All of these algae except *Chlorella* have sharp projections that maximize surface area per unit mass and are able to survive in the presence of predators. Filamentous algae are predominately blue green algae such as *Oscillatoria*, *Anabaena*, and *Phormidium*. *Anacystis* is also a colonial form of blue green algae found in oxidation ponds. *Navicula* is the most common diatom. The filamentous algae and the colonial algae survive primarily because animal predators remove the flagellated algae and *Chlorella*, leaving these undesirable forms to predominate.

The free swimming protozoa *Paramecium*, *Glaucoma*, *Colpidium*, and *Euplotes* have been observed in oxidation ponds. *Vorticella* is a common stalked ciliated protozoa found occasionally. Rotifers and the crustaceans, *Daphnia*, *Moina*, and *Cyclops* represent higher animal predators that can quickly remove the motile green algae and *Chlorella*. These large predators metabolize the available algae and then starve to death, creating a cyclic pattern of growth, death, and regrowth. The undesirable algae predominate when the predators reach peak population since the predators do not metabolize these algae. For this reason some ponds become essentially clear when the crustaceans predominate but other ponds appear to remain green.

Temperature

Temperature is an important parameter affecting oxidation ponds. Not only does temperature have a normal effect on the rate of metabolism of the different groups of microorganisms in the oxidation pond but it has a special effect when the temperature changes suddenly. As the temperature changes normally, the rate of metabolism changes by a factor of 2 with each 10°C temperature change. In the warm summer periods, the rate of microbial growth increases for the bacteria, the algae and the higher animals. As long as the temperature change is relatively slow, there is no problem. With a sudden change in temperature, either up or down, there are short term problems. The reasons for the problems lie in the differential growth rates of the various microorganisms. The bacteria grow the fastest and respond more quickly to the temperature changes. This means that the rate of oxygen demand increases faster than the rate of oxygen supply when the temperature rises. The algae are larger organisms and respond slower. If the bacteria respond too rapidly and create excess turbidity or form black sulfide precipitates, the growth of the algae is retarded even more and the adverse impact is extended even further. Recovery depends upon the growth of the algae and the production of oxygen. A sudden drop in temperature can slow the algae down sufficiently that they settle out before reacting as they should. In the fall a sudden frost will result in the algae dropping out because the low temperature is accompanied by less wind action and more quiescent conditions. The entire pond becomes quite clear. This is the time that regulatory authorities recommend drawing down the pond, since the water quality is at maximum. While many engineers feel that little action occurs during the winter period, this is not entirely true. Microbial reaction continues at a slow rate. The clear oxidation pond begins to turn green even under an ice cover. The response is related to light and temperature. By spring there is a reasonable biological activity but the melting of the ice cover is generally accompanied by too rapid a surface temperature increase. The accumulated organic matter that remained unstabilized after the long winter period stimulates the rapid bacteria growth and the unpleasant anaerobic conditions that persist for a few weeks. When the temperature change is gradual and the ice melting is accompanied by a lowering temperature, the adverse conditions are minimized. Like it or not, sudden temperature changes always bring major changes for oxidation ponds.

Evaporation

The large surface area of the oxidation ponds offers a real opportunity for evaporation. In the eastern part of the United States the rate of rainfall exceeds the rate of evaporation and actually dilutes the effluent. As oxidation ponds move westward, the rate of evaporation equals the rate of rainfall; and then the rate of evaporation exceeds the rainfall. In the far west the rate of evaporation exceeds rainfall many times over and requires a deep pond system with minimum surface area.

It would seem that a zero discharge system would be considered the most desirable. There is no doubt that many small oxidation ponds can be designed as zero discharge systems with the rate of evaporation just equal to the rate of inflow. At the same time, it should be recognized that zero discharge systems accumulate all the salts and inert solids that enter the oxidation ponds. Eventually, the zero discharge system must be abandoned as being unsuitable for biological life and returned to the environment. While such eventuality will be a considerable period in the future, it must be faced. Overall, the continuously discharging oxidation pond offers the best system for pollution control for small communities and industries that require maximum wastewater treatment with minimum effort.

Design Criteria

Oxidation pond design is still a retention time problem. Most plants are designed on conventional concepts but operate at much lower loading rates. The reasons for this are simply that we tend to overestimate both the wastewater flow and its BOD_5 when we apply a general overall design factor. It is easy to apply a single factor, and since it is conservative the engineer continues to use it. Oxidation ponds can play a real role in wastewater treatment for small systems provided they are properly designed. It should be recognized that a single pond system is not satisfactory as it tends to short circuit both algae and fecal coliforms. A multicell oxidation pond system is the only thing that will produce the desired results. It is important that we recognize that a three-cell system will probably provide the best system. A 120-day retention period, three equal cell system with submerged transfer pipes should provide an effluent suitable for EPA secondary treatment requirements. There may be a few days in the summer when the suspended solids and BOD in the effluent exceed the 30-30 standards but the impact on the receiving stream will not be great enough to warrant serious reaction against the treatment system. In the late winter or early spring the fecal coliform criteria may be exceeded for a few days, but once again there is no real evidence that deviation from the fecal coliform criteria represents any hazard whatsoever. It is important that standards reflect real conditions and hypothetical conditions.

Oxidation ponds are simple devices that are based on sound scientific principles. Oxidation ponds offer no magic solutions or fancy technological developments. They are economical to build and to operate. No one wants to collect data on their operating characteristics. No one wants to accept responsibility for oxidation ponds. This is a highly sophisticated era which is more concerned with technological development than with solving real problems. It should be recognized that oxidation ponds can treat wastewaters adequately to meet the needs of the environment if engineers will use the knowledge they already have.

The discharge of raw wastewaters to oxidation ponds results in the discharge of settleable solids that will readily settle out when the velocity drops below minimum levels. The initial discharge of wastewaters should be to a deep section 5 to 10 feet deep and below the normal floor level of the oxidation pond. The solids should accumulate around the influent pipe and create a special anaerobic zone that removes most of these solid materials from the oxidation pond reaction. The extra depth in a center section also results in keeping some of the anaerobic end products from entering the aerobic zone of reaction. There is nothing special about shallow oxidation ponds except where zero discharge ponds are desired or where light is limiting.

Evaporation is a major factor in oxidation pond operation and misinterpretation of the operational data. Evaporation concentrates the residuals and produces a greater concentration effect than would have been produced if evaporation had not occurred. An effluent standard based on concentration alone fails to consider the influent volume and requires an effluent quality that is difficult to obtain even with the most sophisticated system. It is important that we develop criteria that account for evaporation and produce the effluent necessary to prevent significant environmental pollution.

Conclusions

Oxidation ponds have proved over the years to be one of the most significant forms of economical wastewater treatment for small communities and isolated industries. Unfortunately, a lack of understanding of the fundamental biochemistry has resulted in considerable criticism of oxidation ponds. The development of a rigid EPA effluent criteria for domestic wastewater treatment systems has also raised a question as to the validity of using oxidation ponds for wastewater treatment systems. When a total evaluation is made of all factors involved, it is apparent that oxidation ponds have a place in wastewater treatment. With proper design using multiple ponds and adequate holding time and properly designed transfer and effluent pipe structures, oxidation ponds will produce a satisfactory effluent when consideration is made for evaporation over such a large surface area. In a few instances the application of rock filters or slow sand filters may be necessary to produce a high quality effluent; but a high quality effluent can be produced on a continuous basis. The question is not how should oxidation ponds be used, but rather how can engineers improve oxidation pond design to meet effluent criteria and still maintain their economical design and simplicity of operation.

CONSTRUCTION PROCEDURES AND REVIEW OF PLANS AND GRANT APPLICATIONS

W. R. Uhte[1]

General Problems with Wastewater Stabilization Ponds

Before getting down to the nitty-gritty on the design, construction, and operation considerations which are involved in federal and state review and approval of upgrading wastewater stabilization ponds, let's first discuss some of the general problems usually connected with such facilities. As most pond installations are the equivalent of small plants some of the introductory remarks of Professor George Tchobanoglous in his paper on "Wastewater Treatment for Small Communities" given at the Conference on Rural Environmental Engineering, Warren, Vermont, September 1973, are directly applicable to this discussion.

Design

Design of wastewater stabilization ponds for smaller communities is an area that has not received the attention it requires or deserves. Many times large consulting firms will pass up the design of such facilities or assign them to young, inexperienced engineers. In other cases, small firms with little or no experience have undertaken the design of these ponds. Consequently, many of them have now proven to be inadequate to meet original requirements and therefore have no chance of meeting the more stringent discharge requirements established by various federal and state agencies. Clearly, the design of ponds that work, is the responsibility of the engineer and not the contractor or owner.

It must be pointed out, however, that design fee schedules and allowable design costs have effectively perpetuated this dilemma. A small complete wastewater stabilization pond will require just about the same engineering design effort as a large pond. Unfortunately most owners and often even the federal and state regulatory agencies do not recognize this fact and refuse to accept the high engineering costs for the smaller units. Technology transfer information can be helpful, but full recognition of need for reasonable fees would also upgrade design.

Operation and maintenance

Most wastewater stabilization ponds are built and then practically forgotten. If they are visited more than once or twice a year the owner complains that it is a problem installation. Although the more stringent discharge requirements will be a great impact on the need for better operation and maintenance an even greater impact on manpower requirements will result simply from the requirement to produce continuing operational data. Recently it was indicated that the effluent monitoring requirements for a small, less than 1/2 acre, pond installation amounted to 4 man hours per day. This, of course, included two men for safety, travel time and the necessary waiting time between consecutive coliform samples but not laboratory time. It is probably typical for the time which will be necessary to meet the monitoring requirement at most installations.

Once the owner has accepted this situation it will not be difficult to see that much of the other routine maintenance around the ponds also receives attention. Good pond design should include the selection of equipment which will reduce the work associated with such operation and maintenance to a minimum. Such equipment should be well proven, with adequate service and parts centers available within reasonable distances in case of breakdowns.

Budget limitations

With water pollution such a topical item it has been fairly easy lately to convince the voters that capital improvements to the waste treatment system are worthwhile. This is especially true with federal and state participation covering up to 87½ percent of the costs. However, operation and maintenance of the wastewater stabilization ponds have usually been put on the welfare side of the budget; *i.e.*, the side which doesn't produce tangible results such as a library or city park. Unfortunately this intangible side of the budget is the first in line when cuts are to be made in annual expenditures. Often such wastewater facilities are put on emergency budgeting and only when they get into trouble with the local pollution control agency does the owner seriously consider their budget.

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Probably the greatest challenge to the upgrading of wastewater stabilization ponds is going to be in the development of adequate annual operation and maintenance budgets. These receive no federal or state aids, will be literally hundreds of times the costs of existing budgets and will face a long history of being considered an intangible budget item. Only by careful evaluation of the grant requirements for proper operation and maintenance, effluent monitoring and fiscal solvency will this challenge be met. The following discussions of detail design, construction and operation considerations must keep in mind the ever present need to assure that whatever is accomplished under capital improvements be fully implemented on a continuing basis.

Design Considerations

As indicated earlier neither the contractor nor the owner can do much about the design of wastewater stabilization pond. This task is the responsibility of the design engineer and involves the determination of safe loading levels, proper pond recirculation and configuration, good inlet and outlet hydraulics, adequate scum control, safe dike construction, and the correct location and sizing of ancillary facilities.

Loading levels

Most wastewater stabilization ponds are facultative ponds having depths between 3 and 8 feet. The ponds have two active treatment zones: An aerobic surface layer and an anaerobic bottom layer. Oxygen for aerobic stabilization in the surface layer is provided by photosynthesis and surface reaeration, while the decomposition of the sludge in the bottom layer takes place anaerobically. The carbon dioxide given off by the aerobic bacteria degrading organic wastes in the surface zone and by the anaerobic bacteria degrading the sludge in the bottom zone is reused by the surface layer algae to form algal biomass. This algal biomass produces oxygen to support the aerobic activity and dead cells which settle to the bottom and support the anaerobic activity. Facultative ponds are usually loaded between 15 and 80 pounds of BOD_5 per acre per day, although special seasonal loadings of up to 150 to 175 pounds of BOD_5 per acre per day have been found practical.

Other types of ponds are sometimes utilized for special purposes. These include high-rate aerobic ponds, anaerobic ponds, tertiary ponds and aerated lagoons. High-rate aerobic ponds are usually limited to applications wishing to produce a maximum algal biomass, are shallow (12 - 18 inches in depth), and are usually loaded from 60 to 200 pounds of BOD_5 per acre per day. Anaerobic ponds are normally so heavily loaded that no aerobic zone exists. The entire pond is devoid of oxygen with loadings usually ranging from 200 to 1000 pounds per acre per day.

Tertiary ponds are similar to facultative ponds, however they are usually very lightly loaded (less than 15 pounds of BOD_5 per acre per day) and therefore minimize the formation of the heavy algal biomass. They are normally used for polishing effluents from conventional secondary treatment processes. Aerated lagoons derive practically all of the oxygen for aerobic stabilization from mechanical means. No algal biomass is formed so photosynthetic oxygen generation plays no role in the process. Some surface aeration from natural sources does exist, however, aerated lagoons cannot be designed based on surface area loadings. They must be designed utilizing activated sludge parameters for detention and oxygenation capacity.

So far only surface loading rates have been discussed. While these are probably the most useful in determining pond performance, detention time can also be effective. Usually with facultative ponds of reasonable depths (4 to 6 feet) and surface loadings (50 pounds BOD_5 per acre per day) sufficient detention times (90 to 100 days) will be achieved. However, a weak sewage or a system with large quantities of infiltration or inflow can change this situation. An existing pond can be upgraded by either increasing its detention time, decreasing its areal BOD_5 loading or by doing both.

If there is insufficient area for proper pond expansion the ponds may be deepened to achieve the necessary detention time and supplemented with powered mixing and aeration to decrease its areal loading rate. This supplementation is usually achieved by installation of either compressed air diffusers or mechanical aerators. Ponds which require relatively minor supplemental aeration on a uniform basis throughout the year usually find compressed air aeration best. The efficiency of oxygen transfer from air to process BOD_5 demand is usually less than 8 percent. If the ponds must operate the year around in a cold climate, then compressed air aeration will prevent surface freezing and allow direct surface oxygen transfer to supplement whatever photosynthetic activity is available.

When the supplemental requirements are high (from 20 to 50 percent of natural photosynthetic and surface activity) or when the requirements are either seasonal or intermittent, mechanical aerators are used. Two types of mechanical aerators are available: Cage aerators and the more common turbine or propeller vertical-shaft aerators. Mechanical aerators usually have a BOD_5 reduction capacity of between 1.0 and 1.5 pounds BOD_5 per horsepower-hour. The vertical-shaft turbine or propeller units are usually conservatively designed for about 1.15 pounds BOD_5 per horsepower-hour and the cage aerators for about 1.3 pounds BOD_5 per horsepower-hour.

Cage aerators may be used in relatively shallow pond depths (4 to 6 feet), and are usually mounted directly alongside pond dikes. This mounting assures easy access for maintenance and results in reliable above water electric power supply. Mixing energy from the cage aerators seems to have a very large zone of influence. Surface agitation has been measured on photographs over 1000 feet from the aerator. When properly placed near the multiple inlets to a pond the cage aerators have a tremendous pumping and mixing capacity.

Vertical shafted turbine or propeller aerators require greater pond depths (10 to 15 feet) or specially armoured depressed areas under each unit. Pond depth is directly related to aerator horsepower capacity. The units must be mounted within the ponds, thereby requiring special access walkways or boat access and underwater cabling for power and control. Vertical shafted aerators tend to recycle much of their pumped pond volume and therefore seem to have less mixing or agitation influence on the pond surface.

Although the vertical-shafted aerators seem to cost slightly less initially their maintenance and operation costs are usually more expensive than cage aerators. Either type of mechanical aerator may prove cost effective for any given pond installation. Therefore, any comparison should be made giving full consideration of all the facts.

In addition to transferring oxygen to the pond contents, aeration and its resulting mixing also tends to break up thermal stratification which often develops in quiescent ponds. This allows for deeper penetration of sunlight and increases the growth of algae, thereby increasing the pond's capacity for organic loading. Surface agitation from such aeration also keeps the thin surface layer of slick or scum from forming and assures maximum photosynthetic rates and surface aeration.

Wastewater stabilization ponds inherently provide a cost effective means of treatment for excessive summer seasonal organic loadings and winter infiltration and inflow hydraulic loadings. Pond loadings should be conservative to assure this flexibility. Federal grant personnel should be quick to recognize these inherent advantages and minimize the special studies required for the tributary system's infiltration and inflow status. The volume of buffer built into a stabilization pond system eliminates the need for any consideration of shock or diurnal loading variations.

Pond recirculation and configuration

Upgraded wastewater stabilization ponds should include provisions for pond recirculation. Properly designed pond recirculation will involve recycling from four to eight times the average design flow into the system. It should be designed to thoroughly mix the pond contents with the incoming flow prior to its introduction into the pond(s). This pre-mixing provides photosynthetic oxygen from the pond for the immediate satisfaction of the incoming load, assures a completely mixed environment within the feed zones of the pond system, helps to prevent odors and anaerobic conditions within the feed zones, assists in providing pond mixing and simplifies inlet and interpond transfer hydraulics.

The transportation of the recycling pond contents should be via open channels whenever possible. Channels should be of sufficient size to keep velocities low (under 10 feet per minute) and total head loss to a minimum (less than ½ inch) when the system is being operated at maximum recirculation rates. Low lift, high capacity irrigation type propeller pumps usually provide the best efficiencies and lowest first costs for this type of work, although non-clog, self-priming centrifugals and Archimedes screw type pumps are also sometimes used. Regardless of the pumps which are used, however, the pump station should be designed to minimize maintenance and maximize flexibility. If the pump's intake and discharge piping is designed to maintain a siphon condition, pond operating levels can be varied several feet without affecting the pump's capacity or efficiency.

Good pond configuration will use all of the natural and mechanical forces available to achieve full use of the entire wetted area. Inlets, transfer points, and outlets should be located to eliminate dead spots and short circuiting. A proper design will include a study of the prevailing wind directions and will locate outlets in those areas of the pond where surface scum might accumulate. Pond size need not be a limitation if adequate recirculation, distribution, and orientation is maintained.

Often the site or process limitations require multiple pond configurations. This might also result simply from the necessity of upgrading existing facilities. Under these conditions recirculation becomes even more important. When the multiple ponds are in a series configuration, the recirculation reduces the BOD in the mixture entering the first pond and this assures that the organic load is spread more evenly throughout all the ponds. This can be an immediate aid in the reduction of odors, especially where the first pond has been an anaerobic pond. The parallel configuration for multiple ponds is even more effective in reducing pond loadings because its influent is spread evenly across all ponds. In parallel operation recirculation makes distribution simpler for it eliminates the limitations of diurnal flow variations.

Probably the most effective multiple pond configuration with recirculation involves a parallel-series arrangement whereby the parallel ponds are loaded as facultative ponds with recirculation and the final series pond is designed as a natural tertiary unit. This arrangement allows for the final series pond to be operated on an intermittent basis for effluent solids control without affecting the operation of the recirculation system. It also provides a positive separation between recirculating pond contents and the final effluent outlet.

Inlet and outlet hydraulics

Without pond recirculation it is almost impossible to achieve good inlet hydraulics. Such hydraulics must always be designed for peak wet weather flows because this is the capacity of the incoming sewer system. If the influent is pumped into the pond it is usually easier to achieve the proper inlet hydraulics, but often this must be at the sacrifice of good pumping station and upstream sewer system design. Seasonal and diurnal variations will usually mean that for a major part of the time gravity flows will be insufficient to achieve the proper distribution and mixing velocities for good pond feed conditions or the sewage will be stored for increasing lengths of time until sufficient volume has accumulated to operate the influent pumps at their peak flow rate. This can result in system septicity and odor production.

Although most of the literature indicates that ponds should be fed by a single pipe near the center of the pond, this becomes unnecessary when pond recirculation is employed. A single inlet will not achieve full utilization of the wetted area of a pond. Most smaller ponds can utilize multiple entry and single exit systems and assure full utilization. Large ponds (over 20 acres) will usually require multiple inlets and outlets. Even a smaller pond may require multiple inlets and outlets if its configuration and orientation cannot be made to take advantage of its natural flow pattern. When multiple inlet and outlet pipes are used with a recirculation system, inlet and outlet hydraulic losses should be maintained at least ten times the total distribution or collection channel head loss.

If a system must operate over a large variation of flow it should be designed to normally provide good distribution at average daily flow rates and utilize overflow bypassing weirs for those periods when it has to handle peak flows. A good single pipe, multiple port inlet design will result in a 1 foot head loss at average design flow (port exit velocity approximately 8 feet per second). The ports of this single pipe should all be oriented in same direction to induce a mass movement away from the outlet which is usually located nearby. A second pipe with low head overflow weir inlet may be used to accommodate peak flows.

Scum control

Probably the greatest operational problem encountered with wastewater stabilization ponds involves the control of surface accumulations of scum and the odors which develop when these accumulations become septic. A good design will eliminate those areas in which scum normally accumulates and will make it possible for any scum which is formed to be recirculated within the system until it is either degraded aerobically or becomes sufficiently waterlogged to settle to the bottom and decomposes anaerobically.

Control of scum accumulations is best achieved by the elimination of all shoreline or aquatic growths, by proper pond orientation and by the good location of adequate numbers of pond inlets and outlets. Weeds and aquatic growths usually are controlled by periodic spraying, although cutting, sheep grazing and actual physical removal may also be used. Design considerations should assure adequate pond depths to eliminate aquatic growths (usually 4 feet or more) and sufficient shoreline access to make weed control easy.

Earlier design considerations have covered the need for proper pond orientation and the location and number of inlets and outlets. It should be further pointed out, however, that proper transfer pipe design must assure the maintenance of water surface continuity within the entire recirculation system. This means that the pipes must be large enough to limit peak head loss to about 3 to 5 inches with the pipes flowing about two-thirds to three-quarters full. By operating these transfer pipes less than full, unobstructed water surface is maintained between the supply and return channels and the ponds.

Pond transfer pipes are usually best fabricated of bitumastic-coated, corrugated-metal pipe with seepage collars located near the midpoint. This type of pipe is relatively inexpensive, strong enough to withstand rough handling and rapid back filling, flexible enough to allow for the differential settlement often encountered in pond-dike construction, and sufficiently corrosion resistant to assure long life under the proposed wet and dry operating conditions.

Expensive isolating control gates are unnecessary if each of the transfer pipes are accessible from the pond and channels by boat. Such gates can be replaced with specially made fiberglass plugs which can be used to close the pipes to be taken out of service. When such a system is used the design engineer can afford to provide extra transfer inlets and outlets to assure good distribution and then throttle their utilization as required. Hydraulic calculations for partially full transfer pipes require quite a few assumptions, therefore a few extra pipes for flexibility and operation assurance are often well justified.

Dike construction

Pond and channel dikes are usually designed with side slopes between 6 horizontal to 1 vertical and 2 horizontal to 1 vertical. Normally good pond soil will support side slopes of 3 horizontal to 1 vertical. However, a proper design will have the opinion of a qualified soils and foundation engineer to back up its final decision. Safety considerations usually require dike freeboards to be at least 2 and often 3 feet above the pond's peak operating water level.

Dike slopes not only depend on the material available for their construction, but also must consider the wind and water-erosion effects and the protection to be provided. All soils, regardless of slope, need protection in zones subject to turbulence and agitation. Such zones can be created by wind induced wave action, inlet and outlet increased hydraulic activity and aerator agitation. Whenever protection is provided around such zones it should extend several feet beyond the areas involved. Wave protection should extend 1 foot above and below the extreme operating water levels.

Erosion protection can be provided by cobbles, broken or cast-in-place concrete, wooden bulkheads, or asphalt strips. Whatever is used should recognize the need to control shoreline and aquatic growths. The steeper the side slope the less area there is for such protective coverings and aquatic or shoreline growths. Larger ponds in windy areas require heavier erosion protection.

Dike tops should be of sufficient width (10 to 12 feet) to support a 12-inch thick all-weather gravel road. Road surfaces should be crowned to assure quick rainwater runoff and minimum dike erosion. All pond and channel dikes must be provided with such all-weather access roads so that operating personnel may conveniently maintain necessary pond surveillance and control of insects, erosion and plant growth.

Ancillary facilities

Each pond installation must be provided with means for easily launching trailer mounted boats into all water bodies, with means of periodically ascertaining the quantity of influent entering the system, with a method of continuously measuring the quantity of effluent being discharged from the system and with portable grab and composite sampling equipment which can be used to monitor either the influent or effluent on any required cycle.

Boat launching ramps should be built to provide all weather access and good vehicle traction and support for all parts of the ramp above and at least 3 feet below operating water levels. If the level varies several feet, the ramp should extend to a sufficient depth to allow the boat to be launched at the minimum operating level. Ramps should be constructed of concrete, at least 8 feet in width, and provided with a 6-inch curb on the outboard edge.

Influent flow measurements should be made possible by means of pump calibration or manually read flumes designed to minimize flow turbulence and interference. If sufficient head is available a calibrated weir may be used. However, this is a potential odor hazard if the incoming sewage is septic. Periodic influent flow checks should be made to assure that the ponds are not losing flow due to excess exfiltration and to get some estimate of evaporation losses.

The continuous effluent flow measuring system should be designed to provide the most accurate measurement possible without excessive operation or maintenance. Proper design should include the use of accurate V-notch or Cipolletti weirs with simple float actuated electrically timed recorders. Electric timing action should be backed up with a spring power action to assure continuity during power outages. The flow record must be continuous if effluent quantities, treatment efficiencies and total receiving water loadings are to be reasonably determined.

Recent developments in sampler technology have brought on to the market several portable samplers capable of gathering both 24-hour composite or grab samples. With or without electric power available these samplers can provide even the smallest pond installation with the quality measurement capability necessary to satisfy all local, state, and federal water quality requirements. Some samplers even provide for "dry ice" compartments to keep the collected samples chilled.

No write-up on ancillary facilities for wastewater stabilization ponds would be complete without some mention of housing for operation, maintenance, and laboratory activities. Certainly the added treatment and monitoring requirements resulting from the new federal and state regulations will warrant some type of facility capable of supporting these activities. Smaller installations may attempt to contract for these services and thereby hopefully share such facility costs with others. Some may feel that such facilities are best located in the owner's corporation yard. Wherever they are, however, they must reflect the fact that wastewater stabilization ponds are no longer the type of secondary treatment plant which can be built and promptly forgotten. They must now be regularily operated, maintained and monitored.

As a minimum such facilities should contain operator wash and toilet facilities; proper housing for plant growth control equipment; access boat and portable sampling equipment; and compact laboratory capabilities. Minimum laboratory tests should include dissolved oxygen, BOD_5, total solids, suspended solids, volatile solids, and coliform determinations.

Construction Considerations

Once engineering design and local, state, and federal design reviews have been completed a contract for the construction of the wastewater stabilization ponds is put out to competitive bid. In this manner the owner selects the lowest responsible bidder to construct the project. Major state and federal monetary participation dictates that this phase of the upgrading process receives as much attention as the design. Many an excellent design has been sacrificed to loose contract requirements or weak inspection practices. It is very important therefore that these aspects of construction receive careful consideration.

Contract requirements

A good contract begins with a proposal which allows the prospective contractors to clearly understand what they will be obligated to construct and the rules and regulations which will govern and limit their construction procedures. A clear and concise set of drawings and specifications which compliment each other are the keys to this understanding. Additional supporting documents often include soils engineering reports, up-to-date as-built drawings of existing facilities and horizontal and vertical surveying control data.

Drawings must provide the contractor with sufficient information to construct all site and structure improvements in the locations selected. Sometimes on a small contract most of the material quality requirements can also be set forth on the drawings. Most of the time, however, these requirements are reserved for the technical part of the specifications. Regardless of where they are placed, it is always important that both documents compliment and not contradict each other. Contradictions can result in the contractor providing an inferior product.

In addition to the technical specifications a good specification document will also provide the prospective contractor with bidding information, copies of all bidding and contract forms and documents and a complete listing of the rules and regulations governing the project's construction. Bidding information should indicate the routine and special instructions necessary to the submission of a responsible bid. This should include instructions on how the contractor should handle any qualified or variation bids, contingency allowances and major equipment listings. Most sewage treatment facility contracts are best bid as lump sum contracts, especially when the owner cannot afford the extra supervisory expense of assuring the accuracy of unit price quantities. The lump sum contract also makes it possible to assign the complete responsibility of constructing an operable facility to the contractor. If he can prove inadequacy of design to accomplish this, he of course can relieve himself of this responsibility.

Sometimes after reviewing the contract documents a contractor will feel that he can accomplish the same end result by a somewhat different route. If the contract specification provides a perspective contractor a means of submitting an acceptable bid under these circumstances the owner may save money. A good qualified and variation bid clause will do just that.

No engineer can anticipate all of the situations which may develop during construction of a facility. This is especially true with the upgrading of existing facilities. A good design will limit these extra work items to less than 2 to 5 percent of the contract cost, however, depending on the contract size. To assure the prospective contractor that such conditions are going to be expeditiously and properly handled, a contigency allowance of these approximate amounts is included within his bid. If the extra work does not materialize then the money is never spent.

Major equipment listings are a means of assuring that the owner receives his major equipment at the best price available and that the successful contractor will not be able to shop around for lower prices after he has been awarded the contract. When used, such listings must meet the federal regulations requiring the listing of two or more acceptable manufacturers with space for the contractor to write-in other manufacturer's considered "or equal."

By including all the necessary bond forms and other contract documents within the contract specifications the owner eliminates the risk that some proposed contractor will submit a bid with an unacceptable guarantee bond or sign a contract and submit unacceptable material and performance bonds. It also makes the conditions of bidding and being allowed to perform completely and equally clear to every interested contractor.

The rules and regulations governing construction procedures must be mutually equitable if the contract is to stand up in any court test. This part of the contract specifications must be written primarily for the protection of both parties. Neither the owner nor the contractor should be required to be deprived of their basic rights. The owner has a right to expect a finished product within the time specified which meets the quality requirements listed. The contractor in turn has a right to select the method of producing that finished product within the limits of any listed restrictions and to be paid for his production in this manner listed.

Any encroachment by either party into this basic relationship can weaken its entire premise. The owner should not have the right to stop work, unless the contractor is in flagrant breach of contract, and the contractor must operate within the limits set forth in the specifications. If the owner feels he is getting unacceptable quality or if the work is falling behind schedule he should withhold contract payments, not shut down or speed up the work or in any other manner encroach upon the contractor's methods of operation.

The contractor must also realize that the limits set forth cannot be exceeded. Two examples of such limits are the bypassing of existing treatment facilities and the providing of necessary equipment maintenance instruction. Monetary penalties should be spelled out within the contract specifications for bypassing with gradation for degrees and times involved. These penalties should reflect the effect on the owner's reputation and any possible penalties imposed by a regulatory agency. They should be sufficiently expensive to force him to make no bypassing a major consideration in his methods of construction. Equipment maintenance instructions are required prior to the placing of the equipment into operation and for the proper preparation of operation and maintenance manuals. To assure their delivery the contractor should be informed that once the 75 percent level of completion has been reached no further progress payments will be made until all such instructions have been submitted and found acceptable.

Inspection practices

Often it is assumed by the owner that once the engineer has completed the design drawings and specifications he need no longer take an active role in the completion of the project. This is most often true of those owners who already have a construction inspection department overseeing other types of projects under their jurisdictions. Unfortunately this is also sometimes quietly accepted by the engineer, because he realizes that under such an arrangement it will be far more difficult for the owner to make any charge of legal liability hold up in court.

To assure that the design engineer accepts his full responsibility for a project it is necessary for him to also have the full responsibility for the resident engineering and inspection of the construction of that project. The review of shop submittal drawings of process equipment and piping layouts provides the design engineer with the final opportunity to visualize the system's operation and correct any error which has heretofore escaped detection. In addition, the continuous involvement of the design engineer assures that those basic decisions which are often inherent in a design are also reflected in the quality of the final construction.

For wastewater stabilization ponds the major responsibility of inspection will involve the supervision of soil compaction as part of the construction of pond and channel dikes and the assurance that vertical control (elevation) tolerances are within contract requirements. Good inspection requires that an inspector be present whenever the contractor is doing any work on the project.

Operation Considerations

As major contributors to the cost of the construction of the upgraded wastewater stabilization ponds it seems only natural for federal and state environmental protection agencies to have a great interest in their performance. Present federal and some state regulations already require all grant supported plants to have an operation and maintenance manual prepared by the design engineer, to require its personnel to take part in operation training programs, to receive periodic operation performance evaluations by agency representatives and to have developed a system of user charges (or equivalent) sufficient to cover all annual costs, including capital cost recovery.

Operation and maintenance (O/M) manuals

To be of real value to the operation staff of the new or upgraded facility the O/M manual must provide them with a complete description of (1) the treatment processes involved, including its idiosyncrasies, and its normal reaction to stress and seasonal changes; (2) the physical layout, including hydraulic flow pattern, equipment operation and monitoring and control devices; (3) process testing, including laboratory analysis both for control and monitoring treatment efficiencies and results; (4) maintenance requirments, including routine preventive and corrective programs; (5) safety restrictions and precautions; and (6) emergency operation procedures. The manual must be presented in words and pictures which are understandable to lay personnel and should be prepared as soon as practical so that it is available prior to plant set-up. Some parts of the write-up could be valuable early in design, but unless the engineer is assured of adequate remuneration there is little incentive to take the time or make the effort involved at this stage of the process. Some form of up-dating should be made after the plant has operated through a complete seasonal cycle.

Many problems can befall those unwary engineers who assume that this is a simple and relatively inexpensive task. Unfortunately it is similar to the design costs previously mentioned, in that small installations will often require as much or more time and expense than large installations. If O/M manuals are to become the backbone of successful, efficient operation, their true value and cost must be recognized and accepted by all owners and federal and state personnel. These costs can amount to sums which are from 1 to 10 percent of the facilities construction cost depending on whether the plant is a major multimillion dollar or a minor hundred thousand dollar facility. These costs may seem high when compared to normal design fees, but when compared to the accumulated annual cost savings a good manual can accomplish, they become practically insignificant. Federal and state insistence for quality manuals and their willingness to share in the costs of their preparation is one way they can make it easier for the owner to meet the annual budget challenge.

Operation training programs

In most instances owners of upgraded wastewater stabilization ponds will not be able to hire trained experienced personnel to perform their plant's necessary operation and maintenance tasks. Consequently there is, and will continue to be, a great need for operator training programs. Federal and state involvement in the development and implementation of these programs is essential.

On-the-job training with mobile classrooms provides the most effective programs for the smaller treatment plants. This is especially true of stabilization pond operation. This makes good sense because of the need to minimize annual budget impacts. Pond owner budgets do not have room for travel or away-from-the-job expenses. In addition, in many cases the treatment process or configuration for individual plants varies so greatly that the most efficient training for unskilled lay personnel is to train them for specific tasks and not general concepts.

Such on-the-job training should incorporate the use of the plant's O/M manual as much as possible. In most instances, if the specific O/M manual for the facility was systematically studied and thoroughly explained to the plant's personnel the training would meet its goals. Such a review would also provide the owner with an outside opinion of the adequacy of the manual, assure its completeness, and present the federal or state supported trainer with a golden opportunity for developing favorable public relations for the whole environmental protection effort.

Performance evaluations

In many areas performance evaluation programs are just getting started, but already they have raised questions regarding their ability to gather truly representative and meaningful data. Certainly plant performance, housekeeping and maintenance can be observed and meaningfully recorded. When it comes to the determinations of operator training and competency, however, observers must be careful not to accept at face value any single group's viewpoint. Data collected during these evaluations must reflect a consensus of the complete staff. This complicates the data gathering and probably increases its cost, but without such meaningfully developed data these evaluations could easily [illegible] in conclusions and reactions which are completely unverifiable.

Some type of public relations program should be instigated to assure that all plant personnel are aware of the performance evaluation program and what it is set-up to accomplish. Its positive aspects of determining national, state and local needs and the importance of sharing of meaningful data should be emphasized. At no time should the program be allowed to get the reputation that it's just another attempt to develop a technique for "big brother to keep his eye on you."

User charges and self-sufficiency

Federal, and in some cases state, grant regulations already require that grant supported treatment facilities develop and place into action a program of user charges, including a capital cost recovery system, which will assure each system's complete self-sufficiency upon completion of the present upgrading program. Revenue program guidelines further illuminating the restrictions or objectives of this program have been adopted by several states. If upgrading of wastewater stabilization ponds is to achieve its twin goals of improving the discharge from these ponds and providing the data necessary for the documentation of these improvements, then each grant eligible project must have enforced upon itself the necessary user revenue sharing required for this self-sufficiency. Complete information on this user revenue sharing system must be presented to the voters prior to their being asked to accept the capital obligation for upgrading.

Although most financial programs will require the efforts of experts in the public financing field, the owner and federal and state agencies should insist that the design engineer also be actively involved. Preparation of population growth projections, user participation estimates, capital costs and operation and maintenance costs are all areas where input from the engineer is essential.

Summary and Conclusions

Although in many instances this discussion has spoken of design, construction, and operation considerations in general terms most of the comments include specific data, techniques, parameters, and recommendations which when applied judicially to review and approval programs should provide guidelines for judging a project's acceptability. Although it may seem to be the result of an in-born prejudiced or misplaced self-esteem, the one theme throughout the discussion which cannot be ignored is the need for the complete involvement of the design engineer throughout all phases of a project's development. Federal and state regulatory agencies should insist that owners assure this continuity of involvement and take advantage of the responsibility it brings.

POLISHING LAGOON EFFLUENTS WITH SUBMERGED ROCK FILTERS

W. J. O'Brien[1]

Lagoons have been widely used in the United States during the past 20 years for the treatment of wastewater from small municipalities and isolated industrial plants. Their popularity has been based upon relatively low first cost, low operation and maintenance costs, and a high standard of reliability in producing stabilization of the raw wastewater.

The major flaw with lagoons is the algae which are often present in the final effluent. These algae can be removed by a wide variety of coagulation, filtration, flotation, and oxidation techniques. (See McGarry, 1970; Tenny et al., 1969; Shindala and Stewart, 1971; Berry, 1961; Borchardt and O'Melia, 1961; Dodd, 1973; Levin et al., 1962; Parker et al., 1973; Ort, 1972; Folkman and Wachs, 1973.) However, the complexity and the cost of operating most of these processes negate the advantages inherent in the use of lagoons. Submerged rock filters have the potential for avoiding these problems.

Preliminary investigation on the use of submerged rock filters for algal removal was conducted with laboratory scale units by Martin (Martin, 1970; O'Brien et al., 1973). Additional research with pilot scale units located outdoors was completed by Martin and Weller (1973). The findings of these two studies formed the basis for the field scale trials reported herein.

Experimental Facilities

The research filters are located at Eudora, Kansas. The influent for the research pond is obtained from the effluent pipe of the three-cell lagoon system illustrated in Figure 1. This lagoon system is very flexible with regard to the sequence of cells which may be used for treatment. Between June 16, 1973, and February 19, 1974, the facility was operated with cell 2 as the primary and cell 3 as the secondary. From February 20, 1974, to the present time, the system has used cell 1 as the primary, cell 2 as the secondary, and cell 3 for the tertiary treatment.

The sewered population of Eudora is 2,200. The average daily dry weather sewage flow is approximately 208 l/capita/day (55 gpcd), however, there is a large increase in flow during periods of heavy precipitation. The capacity of the lagoon system, when operated with a water depth of 1.52 m (5 ft), is summarized in Table 1. The values given for hydraulic detention time in Table 1 assume plug flow in each cell and no water loss occurs from evaporation or seepage.

Table 1. Physical characteristics of the Eudora wastewater lagoon system.

Cell Number	Surface Area, hectares	Water Volume 10^6 liters	Hydraulic Detention Time, days
1	3.16	45.7	100
2	1.50	21.3	46
3	3.17	45.9	100

The effluent from cell 3 is collected in a wet well and pumped through a 102 cm (4 in) diameter plastic pipe to the research facility. A 5.5 m (18 ft) length of 15.2 cm (6 in) diameter cast iron pipe is used to go through the berm of the experimental lagoon into the inlet structure. The inlet structure divides the flow into two portions and provides for the measurement of each portion with a 15° V-notch weir plate. The lagoon is divided into two parallel basins by an asphalt coated sheet piling wall. Each basin contains an influent pond, a rock filter, and an effluent pond. Flow leaving the experimental facility is collected in an outlet structure which contains two parallel channels equipped with stop planks to control the elevation of the water surface and with 15° V-notch weir plates. Immediately preceding the stop planks are baffles which protrude 76 cm (30 in) below the water surface. The purpose of these baffles is to prevent ice formation in the outlet structure during the winter and to prevent surface scum from leaving the ponds during the summer.

The effluent from the experimental lagoon is collected in a wet well and is pumped back to the inlet structure of the Eudora lagoon system. Plans of

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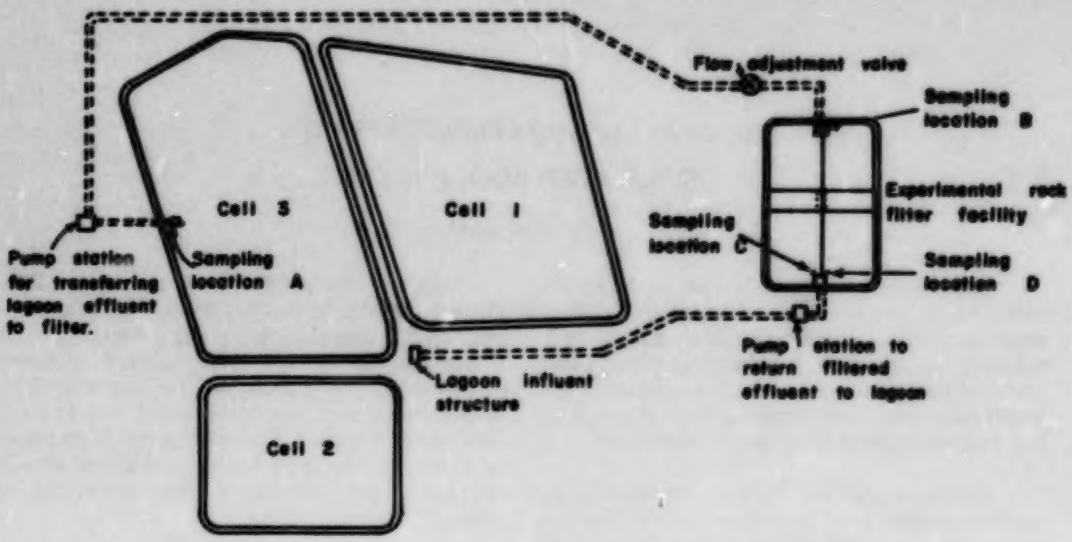
Flow adjustment valve
Sampling location B
Experimental rock filter facility
Sampling location C
Sampling location D
Cell 3
Cell 1
Cell 2
Pump station for transferring lagoon effluent to filter.
Sampling location A
Lagoon influent structure
Pump station to return filtered effluent to lagoon

Figure 1. Physical layout of Eudora, Kansas, municipal sewage oxidation pond system and relative location of experimental rock filter facility (for illustrative purposes only–not drawn to scale).

the experimental lagoon, the inlet structure, and the outlet structure are shown in Figures 2, 3, and 4.

A cross section of the rock filters is given in Figure 5. The filters are constructed of crushed limestone having the size gradations listed in Table 2.

Throughout the remainder of this paper the two filters will be differentiated by designating them as large rock or small rock.

Both filters are 1.4 m (4.5 ft) high and have 1:3 side slopes. The large rock filter is 5.5 m (18 ft) wide at the top, 13.7 m (45 ft) wide at the bottom, and 12.5 m (41 ft) long at the top. The small rock filter is 6.1 m (20 ft) wide at the top, 14.3 m (47 ft) wide at the bottom, and 11.9 m (39 ft) long at the top.

The volume of submerged rock, the volume of the influent and effluent ponds, and the surface area of these ponds when the water depth is 1.22 m (4 ft) are summarized in Table 3.

Each filter also contains three sampling tubes spaced at 2.7 m (9 ft) intervals across the cross-sectional axis. These tubes are constructed from 10.2 cm (4 in) diameter plastic pipe and contain 1.27 cm (.5 in) diameter holes spaced at 7.62 cm (3 in) intervals. There are four holes located 90° apart at each interval.

Table 2. Size gradation of the rock used in the two experimental filters.

Sieve Opening cm	% Weight Retained	
	Large Rock	Small Rock
5.08	7.4	
3.81	28.8	
2.54	52.0	13.4
1.91	10.4	33.1
1.27	1.3	39.0
0.95	0.1	10.4
0.67		3.2
0.47		0.9
Porosity	0.44	0.44

Table 3. Physical dimensions of the experimental filter system when the water depth is 1.22 meters.

	Large Rock	Small Rock
Volume of Influent Pond m^3	126.5	119.4
Surface Area of Influent Pond m^2	165.7	157.6
Volume of Submerged Rock m^3	126.6	142.1
Volume of Effluent Pond m^3	84.3	86.1
Surface area of Effluent Pond m^2	137.1	125.0

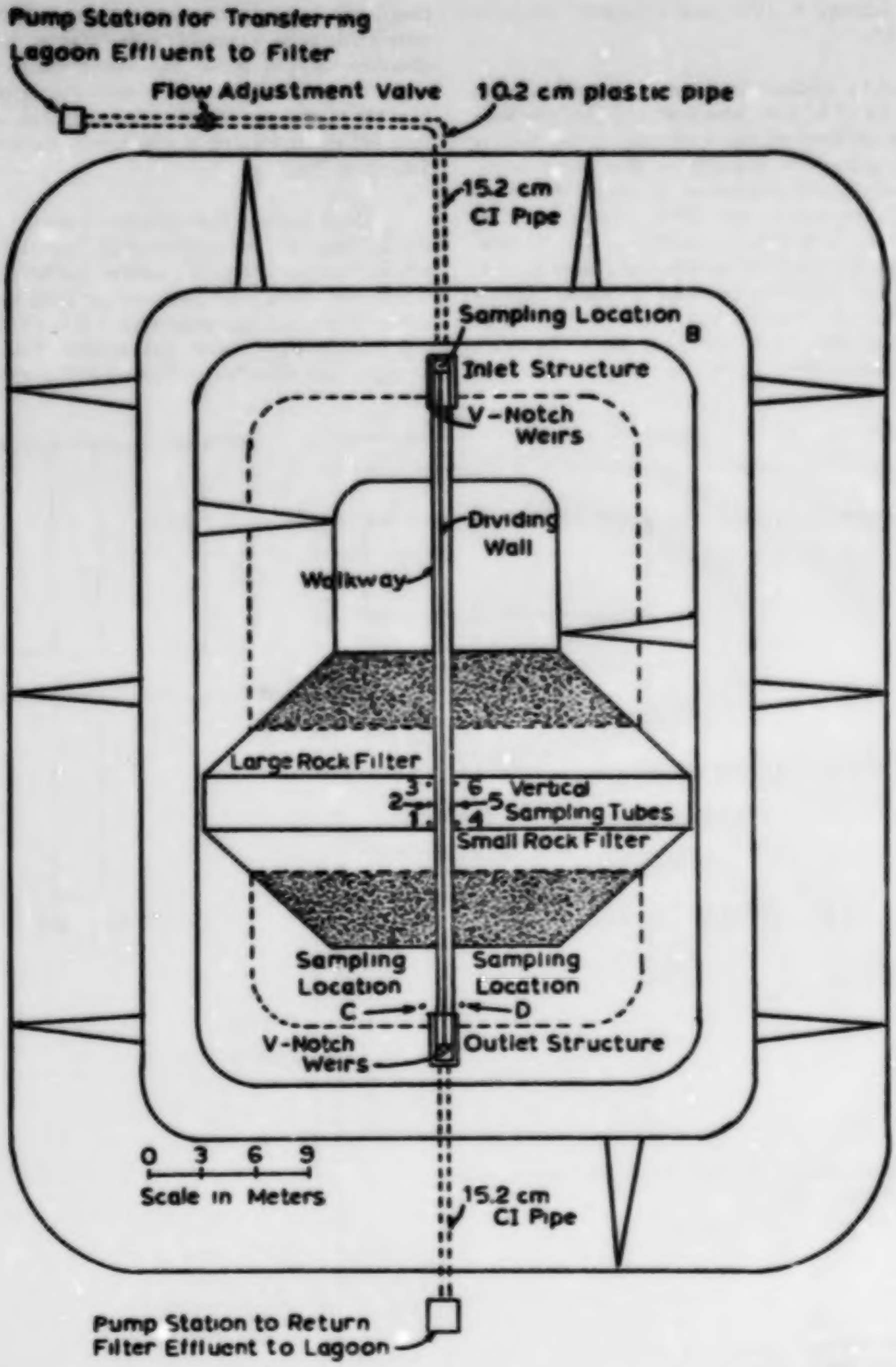
Pump Station for Transferring Lagoon Effluent to Filter
Flow Adjustment Valve
10.2 cm plastic pipe
15.2 cm CI Pipe
Sampling Location B
Inlet Structure
V-Notch Weirs
Dividing Wall
Walkway
Large Rock Filter
2
3
1
6
5
4
Vertical Sampling Tubes
Small Rock Filter
Sampling Location C
Sampling Location D
V-Notch Weirs
Outlet Structure
0 3 6 9
Scale in Meters
15.2 cm CI Pipe
Pump Station to Return Filter Effluent to Lagoon

Figure 2. Plan of experimental rock filter field test facility.

Experimental Program

The experimental lagoon was placed into operation on January 4, 1974, and completed filling on January 10.

Starting January 25 three grab samples, taken between 7 and 12 a.m., have been collected per week from the effluent of the final cell of the Eudora lagoon system, the influent to the experimental lagoon, and the effluent leaving the ponds behind the large and the small rock filters. These sampling locations are illustrated in Figures 1 and 2. Samples were also collected twice weekly from tubes 1, 3, 4, and 6 within the filters (see Figure 2) from April 8 through June 24. Tubes 2 and 5 were sampled twice weekly from May 17 through June 24. All six tubes were sampled on July 12, 19, and 29.

Prior to April 1, in-place temperature measurements were made with a mercury thermometer. No dissolved oxygen measurements were taken. Beginning April 1, *in situ* dissolved oxygen measurements were made with a portable polarographic membrane dissolved oxygen probe and temperature measurements were obtained with a resistance wire probe. The pH of the samples was measured with a laboratory pH meter located in the nearby Eudora Water Treatment Plant.

The samples obtained from the lagoon effluent, the influent to the experimental lagoon, and the effluent leaving the ponds behind the large and the small rock filters were analyzed for total suspended solids (TSS), volatile suspended solids (VSS), total and soluble COD, total and soluble BOD_5, NH_3-nitrogen, and phosphorus. Chlorophyll analyses were

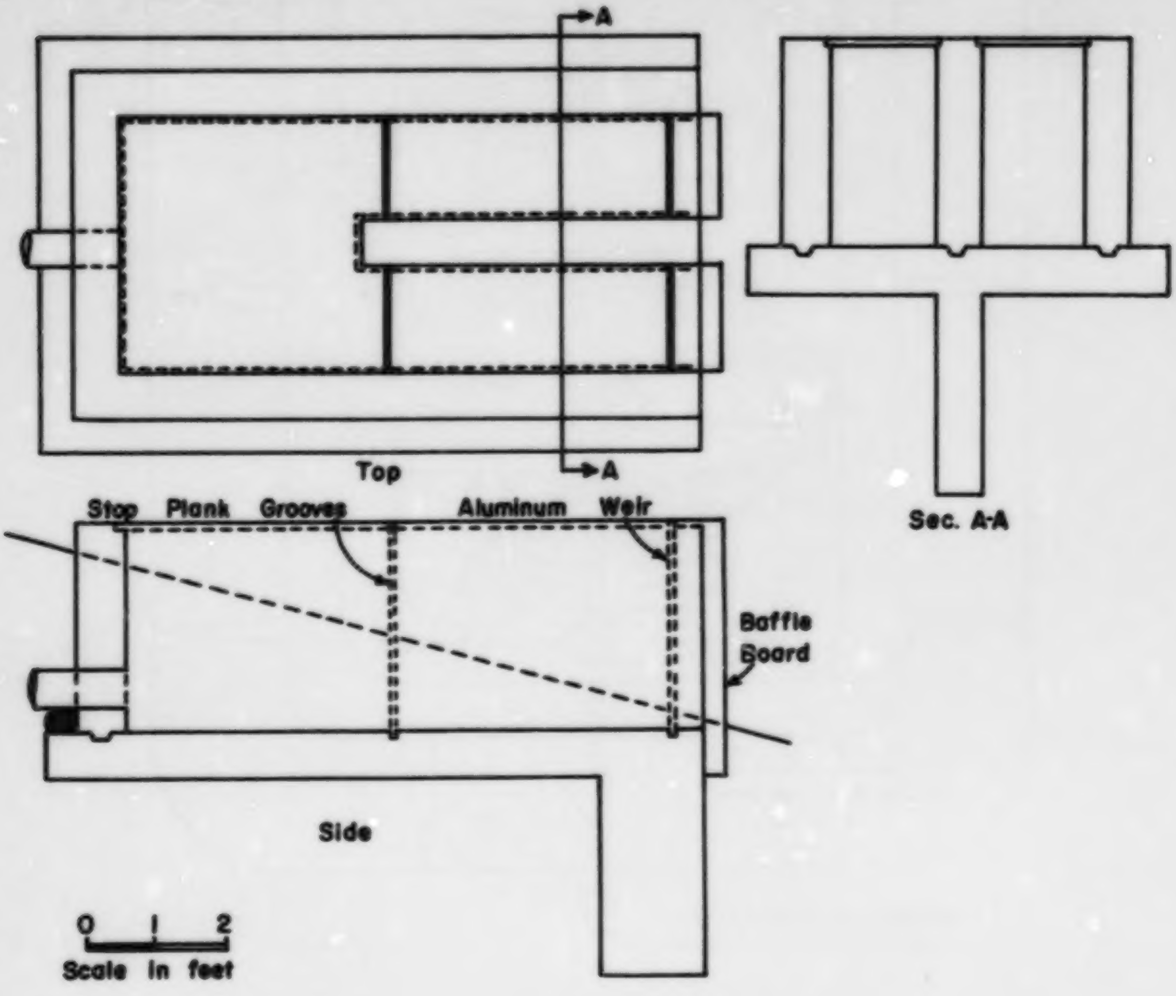

Figure 3. Plan of influent structure of the experimental lagoon.

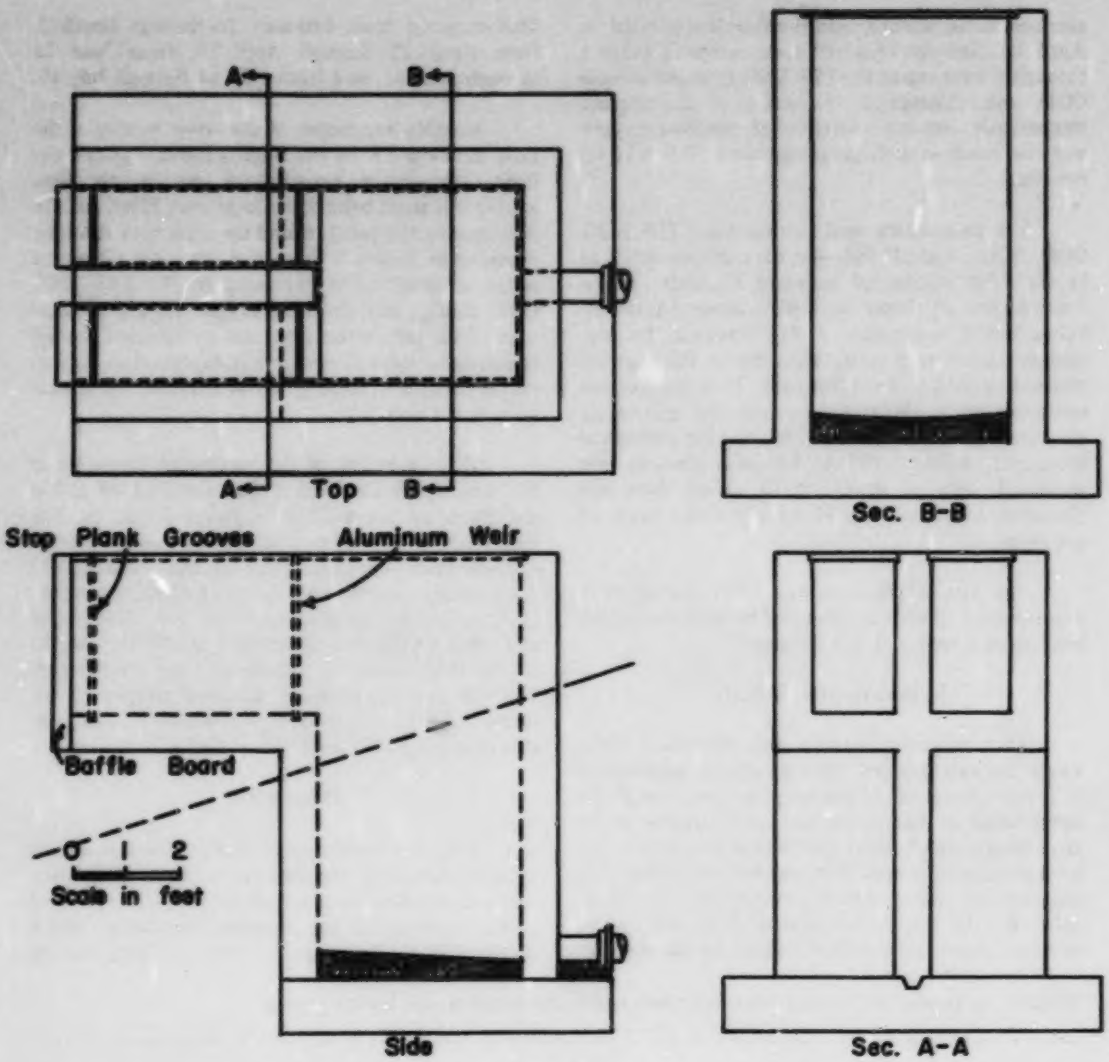
A
B
A
Top
B
Sec. B-B
Stop Plank Grooves
Aluminum Weir
Baffle Board
0 1 2
Scale in feet
Side
Sec. A-A

Figure 4. Plan of effluent structure of the experimental lagoon.

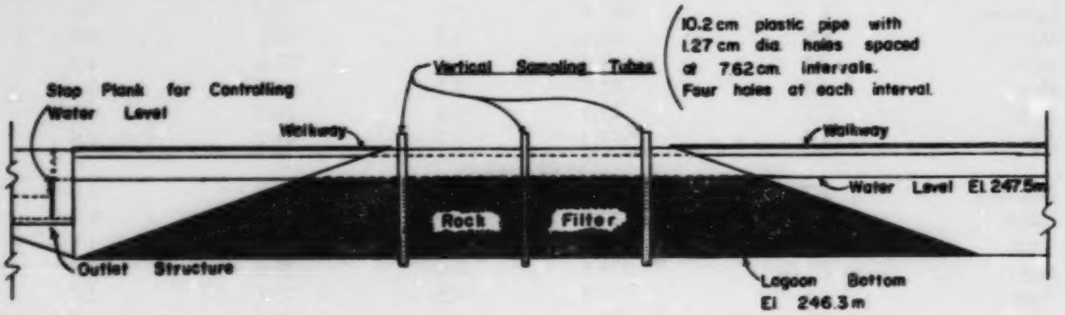
10.2 cm plastic pipe with 1.27 cm dia. holes spaced at 7.62 cm intervals. Four holes at each interval.
Vertical Sampling Tubes
Stop Plank for Controlling Water Level
Walkway
Walkway
Water Level El. 247.5m
Rock Filter
Outlet Structure
Lagoon Bottom El 246.3 m

Figure 5. Cross-section of rock filter.

also conducted starting with the samples collected on April 12. Samples obtained from sampling tubes 1 through 6 were tested for TSS, VSS, total and soluble COD, and chlorophyll. A profile of the in-place temperature and concentration of dissolved oxygen was also made in each sampling tube at 0.3 m (1 ft) intervals.

The procedures used to determine TSS, VSS, COD, BOD_5, and pH followed the methods specified in the 13th edition of *Standard Methods for the Examination of Water and Wastewater* (American Public Health Association, 1971). However, the magnesium carbonate precoat specified in this method was not applied to the filter pads. Total phosphorus was measured by the procedure specified in *Methods for Analysis of Water and Wastes* (Environmental Protection Agency, 1971). Ammonia nitrogen was measured with a Model 95-10 Orion Ammonia Electrode connected to a Model 320 Fisher Accumet pH Meter.

The rate of flow entering and leaving both experimental filters was measured by determining the head on each weir with a point gage.

Experimental Results

The hydraulic loading and detention times within the experimental filter system are summarized in Table 4. These values assume plug flow through the filters which probably does not occur because of the increase in cross-sectional area with depth. Liquid loss from seepage or evaporation was not considered. The time periods during which pumping did not occur, either due to pump malfunction or to the loss of electrical power, were excluded from the calculations. This occurred from February 26 through March 2, from April 22 through April 26, from June 15 through June 17, and from July 14 through July 15.

Monthly summaries of the water quality of the flow discharged from the Eudora lagoon system, the flow entering the experimental lagoon, the flow leaving the pond behind the large rock filter, and the flow leaving the pond behind the small rock filter are provided in Tables 5 through 8. In these tables the water temperature is expressed in °C; TSS, VSS, COD, BOD_5, and dissolved oxygen are expressed in mg/l of the parameter; ammonia is expressed in mg/l as nitrogen; total phosphorus is expressed in mg/l as phosphorus; and chlorophyll is expressed in micrograms/l as chlorophyll.

The magnitude of the parameters measured in the samples obtained from the tubes located within the filters are summarized in Tables 9 and 10. The value used in preparing these tables was the average concentration obtained from samples obtained at four equally spaced depth intervals within each tube. Changes in the concentration of TSS, VSS, total and soluble COD and chlorophyll at different depths in the filters were not significant. The temperature and the concentration of dissolved oxygen in the upper 0.3 m (1 ft) of water was usually greater than that observed in the lower 0.9 m (3 ft).

Discussion

The data presented in Tables 5 and 6 clearly indicate biological metabolism occurred in the wet well and pipeline used to transfer the lagoon effluent to the experimental site. However, the changes which occurred in the water quality were not large enough

Table 4. Hydraulic loading and detention times within the experimental lagoon system.

Large Rock					
		Filter			
Month	Influent Pond Detention Time, Days	Hydraulic Loading		Detention Time, Hours	Effluent Pond Detention Time, Days
		l/day/m^3	gal/day/ft^3		
February	2.7	367.4	2.7	28.7	1.8
March	6.0	165.4	1.2	63.9	4.0
April	2.4	418.9	3.1	25.2	1.6
May	2.0	492.9	3.7	21.4	1[illegible]
June	1.3	743.9	5.6	14.2	0.[illegible]
July	3.9	257.2	1.9	41.1	2.6
Small Rock					
February	2.7	307.8	2.3	34.3	2.0
March	6.7	124.7	0.9	84.7	4.9
April	2.5	339.5	2.5	31.1	1.8
May	2.1	397.6	3.0	26.6	1.5
June	1.4	604.3	4.5	17.5	1.0
July	4.0	210.6	1.6	50.1	2.9

to significantly influence the performance of the submerged rock filters.

The data in Tables 7 and 8 can be subdivided into three intervals of time. The first occurred from the start of operation in January through March 22. During this period the filters functioned primarily as sedimentation basins. However, the capture of suspended solids was relatively low because a biological slime layer had not become completely established on the surfaces of the rock. Algae which settled in the upper portions of the filters were therefore resuspended in the flow moving through lower portions of the filter and eventually washed through the barrier formed by the rocks. This situation was changed by the development of a biological slime layer on the rock surfaces which trapped the algae upon contact. Previous investigations with pilot scale filters indicate approximately 20 days are required to establish a slime layer when the average water temperature is 32 °C (Martin and Weller, 1973). Approximately four months were required to develop a slime layer in this investigation because the temperature of the water was much lower.

The response of the filters during this initial period was also profoundly influenced by the changes which took place within the lagoon system. During December, January, and the first half of February, the lagoons were covered by ice and the ammonia concentration increased appreciably. The ammonia concentration continued to increase until the water temperature increased to approximately 7°C. This triggered an algal bloom which started February 22 and peaked on March 6. The bloom was terminated by a rapid population increase by *Daphnia*. Large numbers of these animals were present in the system until the end of March. Grazing by *Daphnia* eventually lowered the suspended solids concentration in the lagoon effluent to less than 10 mg/l.

The second time interval started March 23 and extended through May 6. During this period the water temperature increased to approximately 20°C and the biological slime layer became fully developed on the rock surfaces. The effect of this layer upon the performance of the filters is vividly shown by the suspended solids and the dissolved oxygen observations summarized in Tables 9 and 10. Throughout

Table 5. Summary of water quality measurements, effluent from the Eudora wastewater lagoon.

Month	Water Temp.	Dissolved Oxygen	pH	TSS	VSS	Total COD	Sol. COD	Total BOD$_5$	Sol. BOD$_5$	Ammonia N	Phos. P	Chlorophyll a + b
February												
Average	5.1		7.6	73	58	132	41	28	7	15.2	8.1	
Range	1.0		7.3	46	45	76	20	15	5	11.5	7.0	
	7.5		8.2	123	77	188	68	39	10	21.0	9.4	
March												
Average	10.2		7.8	66	47	139	56	16	5	12.0	6.5	
Range	6.0		7.4	9	8	88	20	7	3	9.5	4.7	
	15.0		8.5	156	106	208	92	30	8	16.0	9.5	
April												
Average	15.4	11.8	8.0	47	33	99	65	20	5	8.9	6.1	355[a]
Range	11.0	5.7	7.2	25	13	64	52	9	2	5.0	3.4	176
	20.0	16.4	8.6	64	54	140	88	30	8	12.0	10.3	514
May												
Average	22.2	6.6	8.6	54	41	102	52	16	3	0.9	2.7	278
Range	18.9	1.0	8.3	34	29	68	32	8	2	0.1	2.0	142
	25.5	13.9	9.3	89	58	139	68	25	6	4.4	3.5	548
June												
Average	23.2	5.0	8.6	52	40	96	45	13	3	0.15	2.3	176
Range	17.8	2.2	8.3	42	27	82	31	9	2	0.05	1.9	131
	27.0	7.8	8.8	67	52	114	56	18	5	0.21	2.5	228
July												
Average	29.2	5.4	8.8	57	44	99	49	14	4	0.07	1.6	144
Range	26.2	2.6	8.5	42	32	77	36	9	2	0.05	1.3	86
	32.0	7.9	9.1	69	57	136	65	17	6	0.17	1.9	194

[a] Data collected from April 12 through 29.

this period the concentration of ammonia nitrogen in the system was large enough to support a significant amount of algal regrowth in the ponds behind the filters. Regrowth of algae has continued to occur in these ponds throughout May, June, and July, but the magnitude decreased because of the drop in the concentration of ammonia in the lagoon effluent.

The third time period started May 6 and will continue through the remainder of the summer and early fall. It has been characterized by a gradual decrease in suspended solids in the effluent from the submerged filters and by the onset of anaerobic conditions within the filters. The anaerobic environment was first established in the downstream portions of the filters and has slowly moved toward the influent sampling tubes. The influent tube in both filters was anaerobic throughout the bottom two-thirds of the water column on July 29. The concentration of dissolved oxygen in the water moving through the upstream face of the filters will continue to fluctuate throughout the remainder of the summer and the fall. The major variables which determine how much of the filter volume is aerobic are water temperature and the velocity of water entering the filter.

A bright green scum layer consisting primarily of *Chlorella* and numerous species of protozoa and rotifers developed upon the surface of the ponds behind the filters in late April and continued to occur intermittently throughout May, June, and July. The conditions which lead to the development of this layer were apparently the release of ammonia by the anaerobic decomposition of algae in the filters and the relatively thin layer of oxygenated water near the pond surface. The effect of this scum on the effluent quality is not directly reflected in the parameters summarized in Tables 7 and 8 because the baffles on the effluent structure were withdrawing water from the ponds at approximately the mid-depth.

The original objective in providing ponds behind the filters was to increase the concentration of dissolved oxygen and to lower the concentration of ammonia in the final effluent. However, as indicated in Tables 7 and 8, the ponds have not been effective for oxygen transfer and have produced a significant

Table 6. Summary of water quality measurements, influent to the experimental lagoon.

Month	Water Temp.	Dissolved Oxygen	pH	TSS	VSS	Total COD	Sol. COD	Total BOD_5	Sol. BOD_5	Ammonia N	Phos. P	Chlorophyll a + b
February												
Average	4.9		7.6	67	56	147	55	27	6	15.5	7.9	
Range	1.0		7.3	45	38	112	28	15	5	12.5	7.3	
	7.5		8.1	117	87	172	76	37	8	20.0	8.4	
March												
Average	10.0		7.7	63	45	139	57	17	5	11.3	6.4	
Range	5.0		7.4	11	9	76	32	7	3	18.0	3.1	
	17.0		8.1	157	109	220	84	34	8	9.0	8.6	
April												
Average	14.4	5.3	7.9	42	31	113	64	19	5	8.2	5.8	314[a]
Range	10.4	1.0	7.3	20	13	76	52	8	1	4.7	3.4	175
	19.0	8.2	8.5	75	51	160	84	31	8	13.0	7.6	382
May												
Average	21.5	3.3	8.5	48	39	100	49	19	3	0.95	2.8	246
Range	18.1	0.5	8.1	30	23	72	36	9	2	0.05	2.2	114
	25.0	6.3	9.2	72	55	141	76	32	5	3.50	3.3	443
June												
Average	23.8	3.5	8.5	44	34	84	43	12	3	0.18	2.3	162
Range	19.8	1.8	8.3	38	23	69	27	9	1	0.05	2.0	130
	26.5	5.1	8.8	58	44	96	51	17	6	0.80	2.6	200
July												
Average	28.2	0.6	8.6	48	36	104	50	11	4	0.20	1.7	123
Range	26.0	0.2	8.4	33	24	85	38	7	2	0.05	1.3	31
	30.6	1.6	8.8	64	53	144	69	15	6	0.30	1.9	168

[a] Data collected from April 12 through 29.

deterioration in the net removal of suspended solids by providing an environment conducive to algal multiplication prior to final discharge of the effluent. A more effective configuration for the system would be to abut the filter to the lagoon berm. The effluent would be removed through a submerged conduit and aerated prior to final discharge.

A filter system based on this concept has been designed by Lane-Riddle Engineers, Inc. of Higginsville, Missouri, to upgrade an existing lagoon system located at California, Missouri. This installation consists of three cells operated in series. The surface area of each cell at the 1.52 m (5 ft) water depth are: Primary 7.49 ha (18.5 acres); secondary 2.27 ha (5.6 acres), and tertiary 0.77 ha (1.9 acres). The design population is 3600 based on an average flow of 378.51/capita/day (100 gpcd). The rock filter is placed along one side of the existing tertiary cell, Figure 6. The rock used is a dolemictric limestone obtained from a quarry near Jefferson City, Missouri. The size gradation is summarized in Table 11.

The hydraulic loading on the filter is 405.51/day/m^3 (3 gal/day/ft^3). This is based upon the inflow of 378.51/capita/day (100 gpcd) with no adjustment for either evaporation or seepage. Only the rock upstream from the effluent collection pipe is considered part of the filter. The rock located between the pipe and the existing berm is simply fill material. The rock filter is placed upon a 15.2 cm (6 in) thick mat of crusher run base rock. The effluent collection line consists of 48.77 m (160 ft) of double perforated corrugated metal pipe connected to 54.86 m (180 ft) of single perforated corrugated metal pipe. Both pipes are 16 gage metal and are 30.48 cm (12 in) in diameter. The pipes are connected to the existing effluent structure through a series of corrugated metal bends which control the water depth in the tertiary cell at either 0.91 m (3 ft) or 1.52 m (5 ft). All metal parts are coated with asbestos bonded asphalt. The interior and the exterior of the concrete effluent structure is also coated with asphalt. Oxygenation of the final effluent is provided by a cascade aerator located at the end of the outfall pipe.

Construction of the filter at California, Missouri, was done by city employees under the supervision of B. J. Gilbert, Superintendent of Utilities. The construction costs are summarized in Table 12. Obviously

Table 7. Summary of water quality measurements, effluent from the large rock filter.

Month	Water Temp.	Dissolved Oxygen	pH	TSS	VSS	Total COD	Sol. COD	Total BOD$_5$	Sol. BOD$_5$	Ammonia N	Phos. P	Chlorophyll a + b
February												
Average	5.1		7.6	56	48	132	47	21	6	14.7	7.4	
Range	2.0		7.0	41	29	80	24	13	5	12.5	7.1	
	7.5		8.1	84	70	180	72	29	11	19.5	8.0	
March												
Average	10.2		7.9	59	41	110	47	13	4	7.9	6.2	
Range	6.8		7.5	10	8	60	32	7	1	2.3	5.0	
	17.0		8.6	124	94	176	60	29	7	18.0	7.6	
April												
Average	14.8	7.0	8.0	34	26	86	55	19	5	5.0	5.7	206[a]
Range	10.5	1.5	7.4	14	11	60	40	9	2	2.0	3.5	146
	18.9	12.5	8.6	61	51	124	68	32	8	8.5	6.8	305
May												
Average	21.6	0.5	8.4	34	25	76	48	13	4	1.26	2.7	148
Range	18.1	0.0	7.9	17	12	51	32	6	3	0.90	2.4	51
	25.0	2.5	8.8	79	35	97	60	18	7	2.30	3.1	313
June												
Average	24.3	0.1	8.4	24	15	61	43	9	4	1.13	2.1	79
Range	18.8	0.0	8.0	17	6	47	35	7	2	0.60	1.6	58
	26.8	0.3	8.7	31	24	76	56	14	8	2.60	2.6	105
July												
Average	28.4	0.4	8.3	24	15	67	51	9	6	1.49	1.6	59
Range	26.1	0.0	8.0	19	8	53	37	5	3	0.69	1.5	33
	31.1	1.2	8.4	34	25	90	67	13	9	3.00	1.7	83

[a] Data collected from April 12 through 29.

the primary factor in the cost of a submerged rock filter is the filter rock. The design of the California, Missouri, installation is prudently conservative in the selection of both the rock size and the hydraulic loading. When additional information is available to better define the long term operating characteristics of submerged filters, the use of smaller, more heavily loaded units will undoubtedly occur.

A major question concerning rock filters is solids buildup and either clogging or sloughing of material by the system. This is a valid concern because matter is not destroyed. The real question then is, "How soon will the system fail?" Obviously the answer cannot be based upon field experience because filters have not been in use long enough to provide this information. However, the problem may be approached by considering a steady state mass balance. The following assumptions are made:

1. The hydraulic loading rate on the filter is 1081.4 l/day/m^3 (8 gal/day/ft^3)
2. The average influent total suspended solids are 60 mg/l (80 percent VSS)
3. The average effluent total suspended solids are 20 mg/l (80 percent VSS)
4. The specific gravity of the fixed SS is 2.0
5. The specific gravity of the inert VSS is 1.0
6. Sixty percent of the influent VSS is nonbiodegradable
7. The final residue from the influent biodegradable VSS is 20 percent of the initial mass and are inert VSS

Buildup Influent FSS (60-20) (0.20) = 8 mg/l
Influent Inert VSS (60-20) (0.80) (.60) = 19.2 mg/l
Residual Biodegradable VSS (12.8 mg/l) (.2) = 2.6 mg/l

ΔVolume/day

$$= \left| \frac{8}{2} + \frac{2.6}{1} + \frac{19.2}{1} \right| \left(\frac{8 \text{ gal}}{\text{ft}^3} \right) \left(\frac{231 \times 10^{-6} \text{in}^3}{\text{gal.}} \right)$$

$$= (25.8)\,(8)\,(231 \times 10^{-6})\ \text{in}^3/\text{ft}^3 = 0.04768\ \text{in}^3/\text{ft}^3$$

or 17.40 in^3/ft^3/yr.

Table 8. Summary of water quality measurements, effluent from the small rock filter.

Month	Water Temp.	Dissolved Oxygen	pH	TSS	VSS	Total COD	Sol. COD	Total BOD$_5$	Sol. BOD$_5$	Ammonia N	Phos. P	Chlorophyll a + b
February												
Average	5.1		7.6	52	48	137	49	21	6	15.0	7.3	
Range	2.0		7.0	41	39	104	20	13	4	12.0	5.9	
	7.5		8.2	61	58	196	92	32	9	20.0	8.0	
March												
Average	9.9		7.9	46	34	107	52	13	5	9.4	6.1	
Range	6.8		7.5	12	10	60	24	6	2	5.0	5.2	
	14.0		8.7	114	83	160	80	25	9	14.5	7.3	
April												
Average	14.8	6.9	7.9	25	20	86	52	16	6	6.5	5.8	140[a]
Range	10.5	1.8	7.3	15	9	56	40	9	3	4.5	3.1	111
	19.0	13.2	8.4	40	31	120	68	27	12	10.0	7.0	153
May												
Average	21.7	0.6	8.4	34	23	71	49	12	4	1.47	2.6	170
Range	18.5	0.0	8.0	8	8	48	30	5	2	0.80	2.2	56
	25.0	2.0	8.7	81	38	88	64	19	6	2.80	3.0	337
June												
Average	24.5	0.1	8.3	22	14	62	44	8	4	1.49	2.1	63
Range	18.9	0.0	8.0	17	9	53	27	6	3	0.65	1.7	39
	26.5	0.3	8.7	34	27	72	55	12	8	2.70	2.5	92
July												
Average	28.6	0.2	8.3	23	14	65	51	9	5	1.90	1.6	67
Range	26.2	0.0	8.1	17	9	51	41	5	2	0.96	1.4	28
	31.2	0.5	8.5	29	21	100	63	14	7	3.40	1.8	133

[a] Data collected from April 12 through 29.

Table 9. Summary of water-quality measurements obtained from the sampling tubes located in the large rock filter.

Month	Water Temp.	Dissolved Oxygen	TSS	VSS	Total COD	Sol. COD	Chlorophyll a + b
Tube No. 3 Influent Side of Filter							
April							
Average	14.5	2.0	53	44			243
Range	10.7	0.5	33	25			166
	18.5	4.8	79	66			417
May							
Average	21.5	0.6	70	54	106	54	217
Range	18.1	0.2	23	14	80	46	84
	24.7	1.0	122	78	231	61	432
June							
Average	23.5	1.3	48	39	91	48	154
Range	18.6	0.2	40	34	84	42	124
	25.9	2.4	50	42	104	53	187
July							
Average	28.9	1.3	28	20			
Range	27.9	0.2	23	12			
	29.9	2.3	35	30			
Tube No. 2 Center of Filter							
May[a]							
Average	24.7	0.0	27	16			75
Range	24.1	0.0	23	10			71
	25.0	0.0	31	21			81
June							
Average	23.5	0.0	28	20			106
Range	18.4	0.0	24	14			89
	26.1	0.0	32	23			147
July							
Average	29.4	0.2	16	11			
Range	28.9	0.1	13	8			
	30.1	0.2	19	15			
Tube No. 1 Effluent Side of Filter							
April							
Average	14.6	1.5	24	20			136
Range	10.9	0.0	7	4			78
	18.2	8.8	39	35			175
May							
Average	21.2	0.0	29	21	73	53	85
Range	18.2	0.0	19	13	50	44	53
	24.2	0.0	49	35	98	68	146
June							
Average	23.7	0.0	29	17	67	53	79
Range	19.0	0.0	23	10	55	46	58
	25.7	0.0	35	24	84	68	89
July							
Average	29.4	0.3	11	6			
Range	28.7	0.5	9	2			
	30.4	0.1	14	9			

[a] Observations cover the period from May 20-31.

Table 10. Summary of water quality measurements obtained from the sampling tubes located in the small rock filter.

Month	Water Temp.	Dissolved Oxygen	TSS	VSS	Total COD	Sol. COD	Chlorophyll a + b
Tube No. 6 Influent Side of Filter							
April							
Average	14.9	1.8	60	49			278
Range	10.6	0.4	35	26			150
	18.8	3.2	90	78			389
May							
Average	21.6	0.8	122	102	298	46	208
Range	18.1	0.1	31	25	89	39	101
	25.8	1.8	242	208	642	56	386
June							
Average	23.6	3.4	45	37	95	46	168
Range	18.9	1.8	37	29	81	41	132
	26.0	5.2	49	43	112	50	180
July							
Average	29.6	2.0	34	28			
Range	27.9	0.1	31	24			
	30.9	4.1	40	36			
Tube No. 5 Center of Filter							
May[a]							
Average	25.5	0.0	26	17			76
Range	25.2	0.0	19	15			56
	25.7	0.0	32	21			112
June							
Average	24.0	0.0	24	18			74
Range	19.0	0.0	20	15			55
	26.3	0.0	28	24			89
July							
Average	29.8	0.2	12	9			
Range	29.0	0.1	8	6			
	30.3	0.3	14	12			
Tube No. 4 Effluent Side of Filter							
April							
Average	14.5	0.1	16	13			91
Range	10.8	0.0	3	2			63
	18.7	0.3	28	27			142
May							
Average	22.4	0.0	27	19	84	52	109
Range	18.8	0.0	15	11	57	36	45
	25.6	0.0	40	30	138	66	203
June							
Average	24.2	0.0	20	14	71	53	45
Range	19.9	0.0	17	10	63	43	31
	26.4	0.0	23	18	80	68	66
July							
Average	29.5	0.1	12	8			
Range	29.2	0.0	12	5			
	29.9	0.2	13	12			

[a] Observations cover the period from May 20-31.

Table 11. **Size gradation of the rock used in the submerged filter at California, Missouri.**

% Passing Screen by Weight	Screen Size cm
85 - 100	7.62 - 12.70
0 - 15	6.35 - 7.62
0 - 5	below 6.35

Table 12. **Construction cost of the submerged rock filter installation at California, Missouri (August 1974).**

Filter Rock; 4787.90 tons @ $9.50/ton 4342.63 metric tons @ $10.47/metric ton	$45,485.05
Base Rock; 514.30 tons @ $3.69/ton 466.47 metric tons @ $4.07/metric ton	1,897.77
Filter piping	3,675.84
Truck Loader and Operator; 106.5 hr @ $21.00/hr	2,236.50
City Employees; Salary and Wages	1,327.15
Total	$54,622.31

The porosity of both the large and small rock used in this study was 0.44, so the void space is 760 cubic inches per cubic foot. On the basis of the preceding example, it would take approximately 43 years to fill these voids with solids.

Obviously, the steady state assumption used above is an over simplification. During the late fall, winter, and early spring the rate of biological decomposition will be extremely slow because of the low water temperature. There will also be a period during the spring when the TSS will be significantly greater than 60 mg/l due to the nutrients stored in and released from the bottom sediment in the lagoon system. The rate of decomposition during the summer months will be greater than the rate of solids removal from the liquid flowing through the filter during this period. The rate of solids accumulation during part of each year could easily be several times that computed for the steady state system. The effective life of a rock filter will, therefore, be shorter than the value computed in the example. However, because of the extremely low organic loading on submerged rock filters, and the slow rate of solids accumulation within these filters, the effective life should exceed the 20 to 30 design year life used for alternative treatment techniques.

The void space available in both of the rock filters constructed for this investigation was approximately the same. However, the size of the voids is also important because the many small volume voids located between small rocks can be plugged by the layers of biological slime formed on the rocks. This type of failure is expected to eventually occur in the small rock filter used in this study. On the basis of the information presently available, a filter having voids which are large enough to avoid plugging can be constructed from rock having a minimum size gradation approximately equal to the large rock filter used in this investigation and a maximum size gradation approximately equal to the filter installed at California, Missouri. As the rock size gradation increases the maximum allowable hydraulic loading will decrease. However, additional research will be required before a precise relationship between rock size, flow rate, and suspended solids capture efficiency is available.

As shown in Tables 9 and 10, during the initial period of operation most of the reduction in suspended solids occurs in the first 2.74 m (9 ft) of horizontal flow through the rock. The solids capture efficiency of this portion of the filter will decrease with time because the volume of void space will become smaller as inert material builds up and the velocity of flow through this portion of the filter increases. The net effect is analogous to the sedimentation phenomenon observed in slightly inclined tube settlers. In these units the settling zone moves downstream along the axis of the tube as the time of operation increases. Additional research is needed to quantify this response in submerged rock filters.

A second area of concern is the production of hydrogen sulfide by anaerobic decomposition of the algae. In the absence of sulfate, algal decomposition under anaerobic conditions consists of the production of volatile acids and their subsequent reduction to methane. However, if sulfite is present no methane fermentation occurs (Lawrence et al., 1966). A reasonably accurate prediction of the rate of sulfide production can be made by considering the COD change in the system (Foree and McCarty, 1969). The reaction is:

$$8H^+ + 8e^- + SO_4^{=} \longrightarrow S^{=} + 4H_2O$$

In other words, 64 g of organic matter, on a COD basis, will reduce one mole of sulfate and produce 32 g of sulfide ion. The sulfide ion in turn establishes an equilibrium with hydrogen ion to form HS^- and/or H_2S depending upon the pH of the solution.

The data for June for the large rock filter can be used to illustrate the magnitude of the problem. The average total COD entering the experimental lagoon was 84 mg/l and the average total COD in the sampling pipe on the effluent side of the filter was 67 mg/l. Therefore:

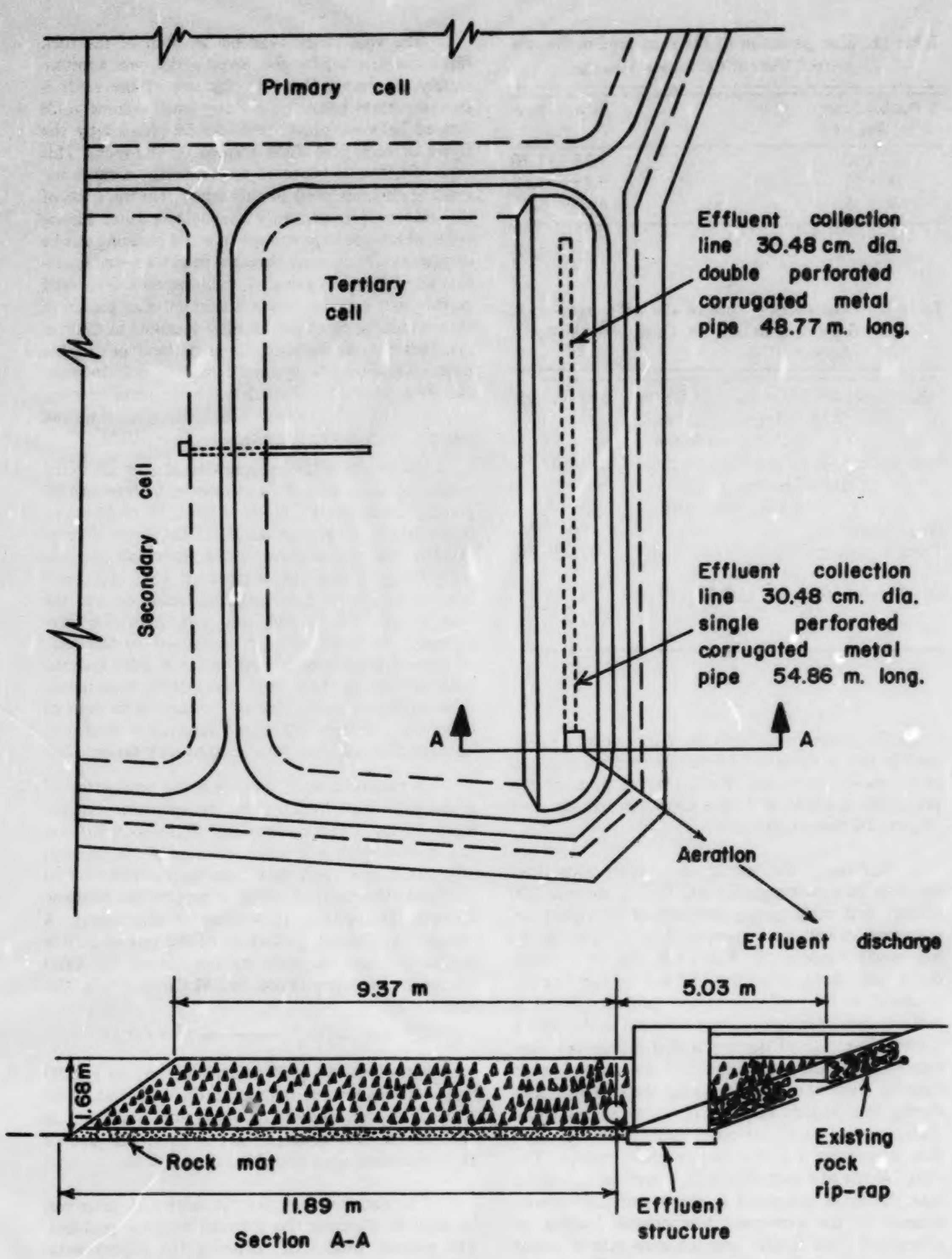
Primary cell
Tertiary cell
Secondary cell
Effluent collection line 30.48 cm. dia. double perforated corrugated metal pipe 48.77 m. long.
Effluent collection line 30.48 cm. dia. single perforated corrugated metal pipe 54.86 m. long.
A
A
Aeration
Effluent discharge
9.37 m
5.03 m
1.68m
Rock mat
11.89 m
Section A-A
Effluent structure
Existing rock rip-rap

Figure 6. Plan of submerged rock filter constructed at California, Missouri.

$$\Delta COD = 17 \text{ mg/l and } \Delta S^{=} = (17 \text{ mg/l}) \left(\frac{32}{64}\right) = 8.5 \text{ mg/l}$$

The total alkalinity of the lagoon effluent at Eudora ranges between 260-275 mg/l as $CaCO_3$. The minimum pH observed within the filters is 7.9. The primary ionization constant of hydrogen sulfide is 1.1 x 10^{-7} at 25°C. Therefore, approximately 10.3 percent of the $S^{=}$ will exist as H_2S (0.9 mg/l) and 89.7 percent will be HS^- (7.9 mg/l). At the present time the odor of H_2S at Eudora is minimal. The intensity of the H_2S odor will be increased somewhat by aerating the final effluent. However, in most locations where lagoon systems are already in use this will not be a significant problem.

A third area of concern is the increase in ammonia nitrogen in effluent from the rock filter. In the pilot scale units this increase ranged from 2 to 3 mg/l as nitrogen. The increase observed in the field scale filters has ranged from 1 to 2 mg/l. It is important to remember, however, that the total ammonia concentration in the effluent is between 1 to 4 mg/l as nitrogen. This is significantly less than the 6 to 20 mg/l of ammonia nitrogen present in the effluent from conventional secondary treatment systems (Barth and Mulbarger, 1966; Fruh, 1967). In locations where the concentration of ammonia nitrogen in the receiving water is critical a very simple adaptation of the submerged filter could be used to nitrify the final effluent (McHarness and McCarty, 1973).

Conclusions

The following conclusions can be drawn from the three phases of investigation that have been carried out on the submerged rock filter:

1. During the first six months of operation the field scale installation has responded in the same way the pilot scale system did during the summer of 1973. Both units went through an initial phase during which the biological slime layer was becoming established on the rock surfaces. During this period the capture of suspended solids was relatively poor. After the biological slime layer was established on the rocks the removal of suspended solids increased dramatically.

2. The biological slime layer will function in an anaerobic environment during part of each year. This will require providing aeration facilities for the filter effluent. This unit need not be elaborate and in many instances a sample cascade aerator will be sufficient.

3. If sulfate is present in the carriage water, hydrogen sulfide will be produced. If the total alkalinity in the lagoon effluent is greater than 260 mg/l as $CaCO_3$ the pH of the final effluent will be high enough to reduce the odor problem to a minimum.

4. The rate of solids accumulation in the filters will be very slow. As long as the size of the void space between the rocks is large enough the prevent plugging the effective life of the filter should be greater than 20 to 30 years.

5. Rock sizes greater than 2.54 cm and less than 12.70 cm will make satisfactory filters. Most of the rock used in any one filter should be within a range of 5 cm in diameter to insure a maximum amount of void volume. As the rock size increases the maximum allowable hydraulic loading will decrease. More investigation is needed to fully define the relationship between rock size and hydraulic loading.

6. Enough information is now available to establish tentative design guidelines for submerged rock filters. These guidelines can be further refined as additional data becomes available.

7. Submerged rock filters are not going to be a panacea for all lagoon installations. However, the results obtained to date indicate these units will be applicable for upgrading the lagoon systems already in operation in many small municipalities.

Acknowledgments

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STABILIZATION POND UPGRADING WITH INTERMITTENT SAND FILTERS

E. J. Middlebrooks and G. R. Marshall[1]

Introduction

Nature of the problem

Many rural communities are still fortunate to be surrounded by large areas of open and relatively inexpensive land. It was because of this land that many of these communities adopted waste stabilization lagoons as a means of wastewater treatment. This treatment scheme requires large tracts of land, but the important consideration was that it gave a satisfactory effluent for minimum cost and maintenance. With the implementation of new water quality standards, a better quality effluent is necessary. If small cities and towns are to economically produce a higher quality effluent, some form of treatment must be utilized that will continue to take advantage of the large areas of relatively inexpensive land surrounding these communities. One method of treatment that capitalizes on the availability of large land areas is intermittent sand filtration.

Objectives

The objective of this study was to evaluate the performance of laboratory and pilot field scale intermittent sand filters as a polishing process that would upgrade existing wastewater treatment facilities. Particular attention was directed toward ascertaining the effectiveness of the intermittent sand filter as a means of removing the highly variable quantities of algae present in stabilization ponds during the warmer months of the year. These results were used to develop preliminary design criteria for intermittent sand filters that would consistently produce an effluent of a quality that would meet stringent water quality standards. Details of this study were reported by Marshall and Middlebrooks (1974).

History of intermittent sand filters

Intermittent sand filtration of sewage began in this country in 1889 in Massachusetts, and for many years was centered in the New England area. Approximately 450 intermittent filter plants were in operation in this country during 1945. But later reports showed a decrease to 398 in use by 1957. Of those still in use by 1957, 94 percent were serving communities with populations under 10,000 (ASCE and FSIWA, 1959).

Successful research efforts at the Lawrence Experiment Station, Lawrence, Massachusetts, resulted in an increase in the use of intermittent sand filters until land costs forced many communities to seek other methods of treating wastewaters. Large numbers of small residential centers such as isolated tourist courts, motels, trailer parks, drive-in theaters, consolidated schools, and housing developments began to spring up all over Florida following World War II. It was soon realized that an economic method of sewage treatment would be necessary for these small communities, and the study of intermittent sand filtration was undertaken at the University of Florida at Gainesville (Calaway et al., 1952; Furman et al., 1949; and Grantham et al., 1949). Much of the modern day knowledge on intermittent sand filters has come out of the studies carried out at Gainesville.

Methods and Procedures

Experimental equipment

The study consisted of both laboratory (Phase I) and field scale (Phase II) experiments.

Laboratory study

Nine laboratory scale filter columns were erected as shown in Figure 1. Each individual filter column was constructed of 6-inch diameter (15 cm) plexiglass cylinders 6 feet (1.85 m) in length. A flanged coupling was provided in the middle of each column to facilitate the filter cleaning operation.

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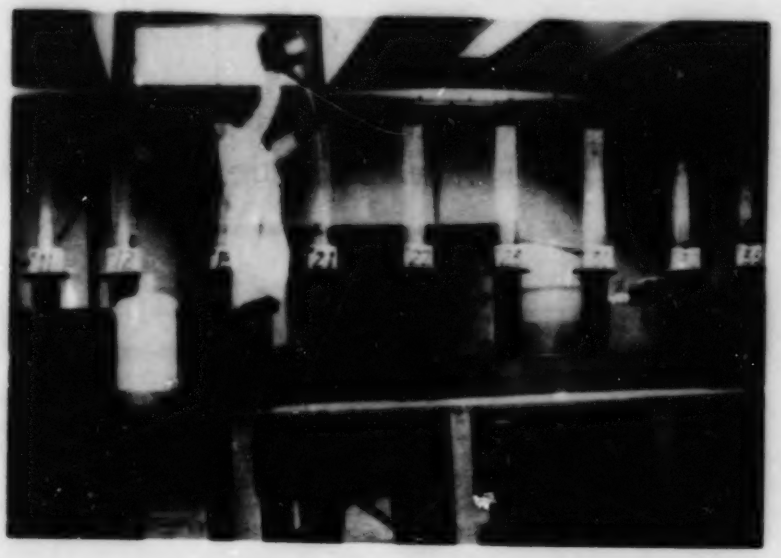

Figure 1. Nine laboratory scale intermittent sand filters shown during daily loading under laboratory conditions.

The filter underdrain material for each laboratory filter consisted of 3-inch layers of 1/4, 3/4, and 1½ inch maximum diameter rock supported on stainless steel mesh. A depth of 28 inches (71 cm) of filter sand was then placed upon the quarter inch diameter rock (6 mm). Sands with effective sizes of 0.17, 0.35, and 0.72 mm and uniformity coefficients of 5.8, 3.8, and 2.6, respectively, were employed.

The 0.17 mm (.0067 inch) effective size sand was the basic sand from which the other two sizes were produced. The sand was a washed bank run sand that was primarily used as fine aggregate in concrete. The 0.35 mm (.0137 inch) sand was produced by sieving the 0.17 mm (.0067 inch) sand through a U.S. series number 50 sieve, the 0.35 mm (.0137 inch) sand being the portion remaining on the sieve. The 0.72 mm (.0283 inch) sand was produced through the use of the U.S. series number 30 sieve.

Logan City, Utah, wastewater stabilization pond effluent was applied once daily to each of the laboratory filters. In order to control the suspended solids concentration in the lagoon effluent applied to the filters, the wastewater effluent was diluted, if necessary, with aerated tap water. Dilution factors were determined on a day to day basis by carrying out a daily suspended solids analysis on the filter influent. Also, prior to dosing, the water temperature was recorded in addition to any other observations noted that day with respect to general filter operation or lagoon performance.

Hydraulic loading rates of 100,000 gpad (153.18 m^3/hectare-day), 200,000 gpad (306.36 m^3/hectare-day), and 300,000 gpad (459.54 m^3/hectare-day) were applied throughout the experiment. Three loading periods of approximately 6 weeks in duration were employed. A loading period constituted a period of operation during which the applied algae concentration was held constant. Plugging is defined as the point in time when all of the specified quantity of wastewater placed on a filter does not pass through the filter in a 24-hour period. Plugging did not occur during any of the three loading periods in the laboratory. At the end of the Loading Periods I and II, the filters were dismantled, the top 10 cm (4 inches) of sand removed from each and replaced with new sand of the same specifications, and the filters were returned to service the same day. At the end of Loading Period III, the top of the sand bed was not removed and daily operation was continued to determine an estimate of the time of operation possible before plugging occurs.

Suspended solids concentrations of 15 mg/l (Loading Period II), 30 mg/l (Loading Period I), and 45 mg/l (Loading Period III) were maintained through each of the loading periods. During the first two loading periods, the wastewater used for filter

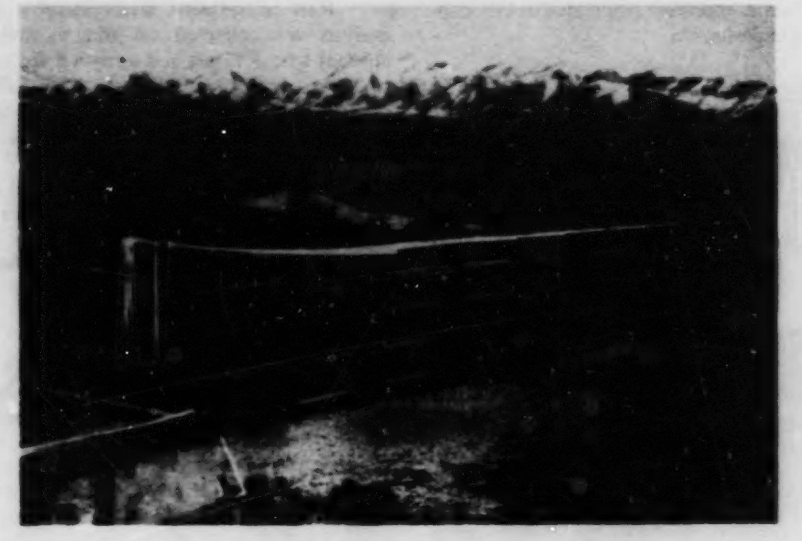

Figure 2. Nine prototype intermittent sand filters located at the point of discharge for the Logan City wastewater stabilization ponds which were used for study under actual field conditions.

loading was obtained directly from the Logan City Wastewater Stabilization Ponds. This water was obtained once weekly and stored under refrigeration for use throughout the remainder of the week. During the final loading period, the influent to the filters was obtained from model stabilization ponds operated in the laboratory. These ponds were enriched with inorganic nutrients and were illuminated on a fixed cycle of 16 hours of light and 8 hours of darkness. In addition, when water was removed each day for filter loading, the sample was replaced with tap water and once weekly the sample was replaced with water obtained from the Logan City wastewater stabilization ponds.

Field study

Nine prototype field filters were erected at the discharge point of the Logan City wastewater stabilization ponds and are shown in Figure 2. These units were 4 feet square (1.2 m x 1.2 m) and 6 feet (1.8 m) in height and were constructed of exterior plywood lined with fiberglass and resin. Underdrain construction was the same as the laboratory filters with the exception being that each of the three layers of gravel were 4 inches (10 cm) in depth.

Six filters each were filled with sands of effective sizes of 0.17 and 0.72 mm (.0067 and .0283 inch) to depths of 30 inches (76 cm). The remaining three units were initially filled with 1/4 inch (6 mm) maximum diameter rock to a depth of 60 inches (152 cm). Later in the study the 1/4 inch rock was replaced with sand of 0.17 mm effective size providing six filters with the basic sand.

Lagoon effluent was applied to the filters with three calibrated pumps operated for a specified period of time. During the fourth week of operation, spreading units were installed to assure better distribution of the raw water on the filter bed. A typical spreading unit is shown in Figure 3.

During the first season of operation, the field filters were also loaded once daily at rates of 100,000 gpad (153.18 m^3/hectare-day), 200,000 gpad (306.36 m^3/hectare-day), and 300,000 gpad (459.54 m^3/hectare-day). The hydraulic loading rates applied in the second season are summarized in Table 1. The filter containing 0.17 mm effective size sand loaded at 900,000 gpad (1,378.62 m^3/hectare-day) was operated at this rate for only 28 days because of the lack of adequate freeboard to compensate for changes in percolation rate due to increased head loss.

A daily sample of filter influent was taken for suspended solids and pH analyses. All other influent parameters were measured on a weekly basis with the exception being the bacteriological samples which were taken immediately following the daily dosing with stabilization pond effluent. No attempt was made to maintain a specified suspended solids content in the field experiments.

Sampling

Laboratory filter effluent samples were collected once weekly and composited from 2 days of operation. Filter influent samples were collected for analysis on the days corresponding to the effluent composite sample.

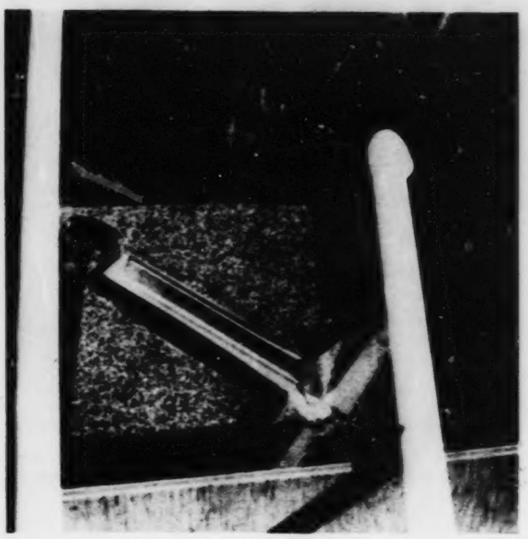

Figure 3. Typical troughs used on the field prototype filters to protect the sand bed and to evenly distribute the applied wastewater over the filter bed.

Raw or influent water samples for bacterial analysis were collected just prior to adding the pond effluent to the filters and analyzed for total bacteria and total coliform bacteria. The following day, effluent samples were then taken and analyzed for total bacteria and total coliform bacteria.

Effluent samples from the field filters were collected once a week and the samples were taken immediately following the application of the pond effluent. A filter influent sample was taken daily.

Analyses

Suspended solids, pH, and temperature measurements were performed on filter influent samples on a daily basis for both the laboratory and the prototype field filters. Filter influent and effluent samples were analyzed once weekly for biochemical oxygen demand (BOD), ammonia, nitrite, nitrate, orthophosphate, total unfiltered phosphorus, suspended solids, and pH. In addition, flask bioassays were performed on the laboratory filter effluent to determine if viable algae cells were in the effluents. Approximately 200 ml of each filter effluent were placed in a 500 ml Erlenmeyer flask and exposed to the lighting pattern described for the laboratory ponds. Growth was measured three to four times weekly in each flask by determining the optical density of the suspension.

Suspended and volatile suspended solids, reactive orthophosphate, and reactive nitrite and nitrate were measured by methods outlined in the Practical Handbook of Seawater Analysis (Strickland and Parsons, 1968). Total phosphorus and biochemical oxygen demand analyses were performed in accordance with Standard Methods (1971). Ammonia concentration was determined by methods described in Limnology and Oceanography (Solorzano, 1969).

Table 1. Physical characteristics of the filters and the hydraulic loading rates applied to the field filters.

Filter Unit Code	Effective Size of Sand		Filter Depth	Hydraulic Loading Rate	
	mm	inches	in	gpad	m^3/hectare-day
A4	0.17	0.0067	30	400,000	612.72
A5	0.17	0.0067	30	500,000	765.90
A6	0.17	0.0067	30	600,000	919.08
A7	0.17	0.0067	30	700,000	1,072.26
A8	0.17	0.0067	30	800,000	1,225.44
A9[a]	0.17	0.0067	30	900,000[a]	1,378.62
C4	0.72	0.0283	30	400,000	612.72
C5	0.72	0.0283	30	500,000	765.90
C6	0.72	0.0283	30	600,000	919.08

[a] Loaded at this rate for 28 days only.

Total plate counts were made in accordance with Standard Methods (1971) with the exception being that all plates were incubated at 20°C for 7 days (Calaway et al., 1952). Total coliforms were determined by the procedures described in Standard Methods (1971).

Results and Discussion

Algae genera

Laboratory filters

Water applied to the laboratory filters were effluent from domestic wastewater stabilization ponds and many different species of algae were present. *Chlamydomonas sp.* was predominant in both the Logan City and the laboratory stabilization ponds. In most cases, the predominant groups of organisms in the filter effluent were "fusiform diatoms." They appeared at one time or another in all effluent samples studied and were observed quite regularly in the applied water.

There were cases where *Chlamydomonas sp.*, *Scenedesmus sp.*, and diatoms were observed separately and in various combinations in the effluents. There were a few effluent samples in which algae were not observed. This usually occurred in the 0.17 mm (.0067 inch) effective size sand subjected to the lowest loading rate. When the lagoon effluents were applied to the filters, the algae, *Chlamydomonas sp.*, were usually found in a "clumped" or palmelloid state and in the effluent were observed to be single, motile cells in nearly every case. This palemelloid state may have contributed significantly to the removal efficiencies obtained with the filters.

Field filters

Table 2 summarizes the results of the microscopic analyses for the field filters. As reported for the laboratory filters, *Chlamydomonas sp.* were again predominant in the filter influent during the second year of the field study. A variety of genera was present in the lagoon effluent, but *Chlamydomonas sp.* represented a minimum of 70 percent of the algal population throughout the study.

Oxidation of nitrogen

Laboratory filters

Ammonia concentrations in the influents and effluents were not measured until Loading Period III. As the applied, effluent and removal values show (Table 3), ammonia was present in large quantities and was readily oxidized. This is in agreement with earlier results reported at the University of Florida where settled primary sewage was applied to intermittent sand filters (Furman et al., 1949; and Grantham et al., 1949).

Table 4 shows the relationship between hydraulic loading rate, sand size, and the effluent nitrate concentration for the three algal concentrations applied (Loading Periods I, II, III). During Loading Period III, the ammonia concentration, Table 3, was found to be high in the artificially produced wastewater stabilization pond effluent when compared with concentrations that would be expected to exist in a tertiary treated wastewater stabilization pond effluent. Thus, the large increase in nitrification observed during Period III, when compared with that of Periods I and II, was probably

Table 2. Algae genera population estimates for the influent and effluent samples from the field filters.

Sample Date	Genera									
	Influent Sample						A5 & C5 Effluent Samples			
		Chlamydomonas								
	Anabaena	Vegetative	Palmelloid	*Daphnia*	Diatom	*Euglena*	*Anabaena*	*Chlamy.*	Debris	Diatom
26 July	0%	25%	70%	[a]	5%	[a]	0%	85% (dead)	Mainly	15%
2 Aug.	0%	25%	70%	[a]	5%	[a]	0%	85% (dead)	Mainly	15%
9 Aug.	0%	25%	70%	[a]	5%	[a]	0%	85% (dead)	Mainly	15%
15 Aug.	[a]	20%	70%	[a]	5%	5%	0%	85% (dead)	Mainly	15%
22 Aug.	[a]	5%	85%	[a]	5%	5%	0%	85% (dead)	Mainly	15%
28 Aug.	5%	5%	80%	[a]	5%	5%	0%	85% (dead)	Mainly	15%
7 Sept.	10%	10%	75%	[a]	5%	[a]	[a]	85% (dead)	Mainly	15%
13 Sept.	15%	10%	75%	[a]	[a]	[a]	5%	80% (dead)	Mainly	15%
19 Sept.	20%	5%	70%	[a]	5%	0%	10%	75% (dead)	Mainly	15%
27 Sept.	10%	[a]	80%	[a]	10%	0%	0%	80% (dead)	Mainly	20%

[a] Occasional.

Table 3. Mean applied and effluent ammonia nitrogen concentrations obtained during Loading Period III in the laboratory study.

Applied NH_4-N (mg/l)	Effluent NH_4-N Concentration, mg/l — Effective Size of Filter Media								
	0.17 mm			0.35 mm			0.72 mm		
	Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}		
	100	200	300	100	200	300	100	200	300
2.13	.006	.004	.006	.006	.014	.017	.043	.146	.217

Table 4. Mean applied and effluent nitrate nitrogen concentrations obtained in the laboratory study.

Loading Period	Applied NO_3-N (mg/l)	Effluent NO_3-N Concentration, mg/l — Effective Size of Filter Media								
		0.17 mm			0.35 mm			0.72 mm		
		Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}		
		100	200	300	100	200	300	100	200	300
I	0.034	1.45	1.25	1.20	0.99	1.12	1.63	1.06	1.02	1.09
II	0.110	0.96	0.91	0.91	0.84	0.81	0.74	0.82	0.71	0.76
III	0.165	4.04	3.57	3.89	3.82	3.44	3.03	3.97	3.17	2.81

caused by the greater amounts of ammonia nitrogen present in the artificially enriched wastewater effluent produced in the laboratory ponds, and was probably not related to the increased applied algae concentrations during Loading Period III.

Filters constructed of sands with smaller effective sizes more readily oxidized ammonia to nitrate (Table 4). This result agrees with the findings of Grantham et al. (1949), Furman et al. (1949), and Pincince and McKee (1968).

Table 3 shows the changes in ammonia-nitrogen concentrations at the three hydraulic loading rates and filter sand sizes. The 0.72 mm (.0283 inch) effective size sand filter showed a slight decrease in ammonia-nitrogen reduction as the hydraulic loading rate increased. This decrease was probably caused by increased submergence, decreased aeration, and a reduction in the contact time within the filter bed.

Field experimental results shown in Tables 5 and 6 were in agreement with the results observed in the laboratory filters. The 0.17 mm (.0067 inch) effective size sand was somewhat more efficient in the oxidation of ammonia-nitrogen than the 0.72 mm (.0283 inch) sand. Ammonia-nitrogen oxidation was not continued in the second year of the field study; however, as the hydraulic loading is increased, a corresponding decrease in oxidation would be expected. The rock filtering media oxidized little of the ammonia-nitrogen to nitrate. This is probably due to the short time required for the liquid to pass through the media.

BOD removal

Laboratory filters

As shown in Table 7, the concentration of BOD_5 in the lagoon effluent applied to the laboratory filters was close to the existing Utah standard of 5 mg/l even before filtration during Loading Periods I and II. This was caused by two factors: The necessity to dilute the effluent to obtain the desired suspended solids concentration applied to the filters, and the

Table 5. Mean applied and effluent ammonia nitrogen concentrations obtained in the field study during the first year.

Applied NH_4-N (mg/l)	Mean Effluent NH_4-N Concentrations, mg/l		
	.17 mm	.72 mm	6 mm max. dia. rock
1.09	0.013	0.426	1.10

Table 6. Mean applied and effluent nitrate nitrogen concentrations obtained in the field study during the first year.

Applied NO_3-N (mg/l)	Mean Effluent NO_3-N Concentrations, mg/l		
	.17 mm	.72 mm	6 mm max. dia. rock
0.078	0.996	1.11	0.479

Table 7. Mean applied and effluent BOD_5 concentrations obtained in the laboratory study.

Loading Period	Applied BOD_5 (mg/l)	Effluent BOD_5 Concentration, mg/l								
		Effective Size of Filter Media								
		0.17 mm			0.35 mm			0.72 mm		
		Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}		
		100	200	300	100	200	300	100	200	300
I	6.71	1.15	1.55	2.31	2.51	2.61	2.97	2.89	3.09	3.01
II	6.34	1.17	1.26	1.96	2.44	2.08	2.41	2.33	2.50	1.93
III	36.5	5.81	5.64	7.14	11.21	10.83	11.53	12.26	12.72	13.25

high degree of BOD_5 removal produced by the 5-stage Logan lagoon system.

Results of the laboratory study were in good agreement with results obtained by Grantham et al. (1949). Examination of Table 8 shows that the loading rates used had little effect on BOD_5 removal. However, the data show a trend toward an increase in the concentration of BOD_5 in the effluent as the loading rate increased. Higher loadings would probably show an even greater increase in effluent BOD_5 concentrations for all sand sizes. With respect to sand size, the effect of loading rate does slightly decrease the filter's ability to remove the applied BOD_5 which agrees with the findings of Grantham et al. (1949).

Field filters

At the same hydraulic loading rates as those employed in the laboratory filters, BOD_5 removals obtained during the first year in the field units in general agreed with the laboratory findings with the exception being the lower removal efficiencies obtained with the 0.72 mm (.0283 inch) sand used in the field (Table 9). The differences in performance summarized in Table 8 were probably caused by the 10-20°F greater operating temperature under laboratory conditions. In general, lower air temperatures produce filter effluents with higher BOD_5. This effect was even more pronounced in larger sized sands studied by Grantham et al. (1949). Coarser sands allow better aeration which would allow the air temperature to exert a much greater effect on the biological activity.

The mean monthly influent and effluent BOD_5 concentrations obtained during the second year of field operation at various hydraulic loading rates for the two effective size sands (0.17 and 0.72 mm) are presented in Table 10. The BOD_5 of the influent remained essentially constant during the second year of operation ranging from 10.0 to 24.9 mg/l with an average value of 13.7 mg/l and a median value of 12.5 mg/l. There appears to be little variation in the effluent BOD_5 concentration with hydraulic loading rate for the 0.72 mm effective size sand; whereas, the 0.17 mm effective size sand shows a definite increase in effluent BOD_5 concentration as the hydraulic loading rate was increased. It is likely that the effluent BOD_5 concentration would also increase for the 0.72 mm sand sizes if the loadings were increased to 0.7 and 0.8 mgad.

The mean effluent BOD_5 concentrations for the 0.17 mm effective size sand filters loaded at 700,000 and 800,000 gpad (1072.3 and 1225.4 m^3/hec-

Table 8. The comparison of BOD_5 removal for the laboratory filters during Loading Period II and the field filters containing 0.72 mm (.0283 inch) size sand.

Hydraulic Loading (gpad)	Percent BOD_5 Removal Under Laboratory Conditions (70°F Ave. Air)	Percent BOD_5 Removal Under Field Conditions (60°F Ave. Air)
100,000 (153.4 m^3/hectare-day)	63.2	24.3
200,000 (306.4 m^3/hectare-day)	59.6	17.8
300,000 (459.5 m^3/hectare-day)	69.6	29.6

tare-day) was twice as high as the values obtained at a hydraulic loading rate of 600,000 gpad (Table 10). A higher effluent concentration was expected, but whether such a large increase would have occurred if all of the filters had operated for an equal time period is unknown. However, based upon the data collected in this study it appears that BOD_5 removal efficiency reaches a limit in the vicinity of a hydraulic loading rate of 600,000 gpad.

BOD_5 reductions with the 0.17 mm effective size sand filters ranged between 38.7 and 97.4 percent with the lower reductions occurring principally at the higher hydraulic loading rates (700,000 and 800,000 gpad). BOD_5 reductions obtained with the 0.72 mm effective size sand appeared to be independent of the hydraulic loading rate and ranged between 27.0 and 80.7 percent.

Table 9. Mean applied and effluent BOD_5 concentrations obtained in the field study during the first year of operation.

Applied mg/l	Ave. Effluent BOD_5 Concentrations, mg/l		
	0.17 mm	0.72 mm	6 mm max. dia. rock
6.18	1.07	4.70	4.92

Mean BOD_5 reductions for the second season for the 0.17 mm sand ranged from 70.4 percent at a hydraulic loading rate of 800,000 gpad to 88.4 percent for the 400,000 gpad rate. Mean BOD_5 reductions for the 0.72 mm sand were essentially constant for all hydraulic loading rates and ranged from 59.9 to 63.2 percent.

Phosphorus removal

Phosphorus was initially removed by the intermittent sand filters, but removal was greatly affected by the length of time that the units had operated and the hydraulic loading rate. Because little biological growth occurs on or in the filter as the water passes through, it is unlikely that any significant phosphorus removal was obtained through growth needs. Therefore, the most obvious explanation of the relatively large phosphorus removals obtained at the beginning of the experiments was ion exchange. The sands contained some forms of carbonate which probably

Table 10. Mean influent and effluent BOD_5 concentrations obtained with each sand size and hydraulic loading rate during Phase II under field conditions.

Month	Mean Monthly Influent BOD Concentration (mg/l)	Mean Monthly Effluent BOD_5, mg/l															
		Effective Size, 0.17 mm										Effective Size, 0.72 mm					
		Hydraulic Loading Rates, gpad										Hydraulic Loading Rates, gpad					
		400,000		500,000		600,000		700,000		800,000		400,000		500,000		600,000	
		mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.
June	12.1	0.75	93.8	1.2	90.1	1.3	89.3	-	-	-	-	4.5	62.8	3.6	70.2	3.9	67.8
July	12.6	1.5	88.1	0.87	93.1	1.1	91.3	3.5	72.2	3.3	73.8	5.4	57.1	5.7	54.8	5.9	53.2
Aug.	12.9	2.2	82.9	3.1	76.0	3.1	76.0	4.2	67.4	4.3	66.7	6.2	51.9	5.8	55.0	6.8	47.3
Sept.	16.1	2.0	87.6	1.7	89.4	1.8	88.8	3.4	78.9	4.7	70.8	5.9	63.4	5.1	68.3	5.4	66.5

served as the exchange medium. Once saturated, there would be no phosphorus removal of any consequence. Phosphorus removals in the field units followed the same pattern observed in the laboratory.

Algal removal

Algal concentrations in the influent were estimated by the suspended solids technique which measures a variety of organisms, inert suspended matter, and a number of various algae species. Effluent algal concentrations were also estimated as volatile suspended solids to overcome the disadvantages of the silts and clays washed from the filters during the early stages of the study.

Laboratory filters

Suspended and volatile suspended solids concentrations applied and in the effluents of the laboratory filters are shown in Tables 11 and 12 for the various hydraulic loading rates and sand sizes employed. The suspended and volatile suspended solids removals were independent of the hydraulic loading rates employed. However, after the silt and clay were removed, it appeared that a general trend was developing which indicated an increase in effluent solids concentration as the hydraulic loading was increased, particularly when greater concentrations of suspended solids were applied.

Suspended solids removals were directly related to the effective size of the sands at the higher solids loading rates. At lower loadings the removals obtained on the 0.72 mm (.0283 inch) sand were approximately equal to the removals obtained with the 0.35 mm (.0137 inch) filters.

Some algae passed through the entire depth of the filter bed as verified by microscopic examination of the effluents. Borchart and O'Melia (1961), Ives (1961), and Folkman and Wachs (1970) have reported similar results. Percent removal efficiencies increased with the application of higher algal concentrations, but more algae passed the filter than at the lowest applied concentration. Flask bioassay results are presented later in an attempt to study the ability of those algae present in the effluent to grow.

Field filters

Algal removals by the field filters during the first year of operation are not shown because the silt and clay that was washed from the filters made interpretation of the results impossible. During the second year of operation, algal concentrations were

Table 11. Mean applied and effluent suspended solids concentration obtained in the laboratory study.

Loading Period	Applied Suspended Solids mg/l	Effluent Suspended Solids Concentrations, mg/l								
		Effective Size of Filter Media								
		0.17 mm			0.35 mm			0.72 mm		
		Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}		
		100	200	300	100	200	300	100	200	300
I	31.0	5.53	7.93	11.2	10.6	10.9	12.8	13.6	11.9	11.0
II	13.7	3.96	4.80	6.05	9.39	8.19	6.50	11.0	8.15	7.28
III	46.3	1.86	1.93	5.33	9.47	11.9	13.7	16.6	15.9	16.5

Table 12. Mean applied and effluent volatile suspended solids concentrations obtained in the laboratory study.

Loading Period	Applied Volatile Suspended Solids mg/l	Effluent Volatile Suspended Solids Concentration, mg/l								
		Effective Size of Filter Media								
		0.17 mm			0.35 mm			0.72 mm		
		Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates gpad x 10^{-3}		
		100	200	300	100	200	300	100	200	300
II	9.16	1.99	2.14	2.30	3.38	3.33	3.40	3.85	4.00	3.17
III	41.3	1.46	1.70	3.48	7.28	7.14	8.31	10.1	13.1	13.2

estimated by measuring fluorescence[2] and by determining suspended and volatile suspended solids. Linear regression analyses of the solids and fluorescence measurements produced a linear relationship significant at the 1 percent level.

Influent and effluent algal concentrations expressed as suspended solids produced by the field filters are summarized in Table 13. Volatile suspended solids concentrations are shown in Table 14. Data for the 0.17 mm effective size sand filter loaded at 900,000 gpad are not presented because of the relatively short period of operation. However, algal removals were similar to those obtained with the 800,000 gpad loading rate.

Figure 4 shows the relationship between the hydraulic loading rates and the suspended and volatile suspended solids concentrations in the filter effluents for the 0.17 and 0.72 mm effective size sands. Algal removal apparently is independent of hydraulic loading rate up to a loading of approximately 600,000 gpad.

Effluent suspended solids concentrations for the months of May and June 1973 were much greater than the concentrations in the effluents. This was attributed to the washing of fines and clay from the filter sand. Filter media were produced from pit run sands containing large quantities of fines and clays that were easily washed from the filters. Clean water

Table 13. Mean influent and filter effluent algal concentrations measured as suspended solids for each sand size and hydraulic loading rate studied during Phase II under field conditions.

Month	Mean Monthly Influent Algae Conc. as Suspended Solids (mg/l)	Mean Monthly Effluent Suspended Solids Concentrations and Percent Removals															
		Effective Size Sand, .17 mm										Effective Size Sand, .72 mm					
		Hydraulic Loading Rate, gpad										Hydraulic Loading Rate, gpad					
		400,000		500,000		600,000		700,000		800,000		400,000		500,000		600,000	
		mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.
May	5.0	25.1	-	56.0	-	20.9	-	-	-	-	-	31.7	-	7.5	-	15.9	-
June	6.5	15.7	-	38.9	-	14.5	-	-	-	-	-	11.6	-	9.4	-	12.5	-
July	29.8	14.2	52.3	23.9	19.8	20.2	32.2	21.4	28.2	15.4	48.3	17.9	39.9	14.4	51.7	16.9	43.3
Aug.	44.2	23.2	47.5	18.8	57.5	30.0	32.1	34.5	21.9	39.1	18.6	33.0	25.3	22.4	49.3	26.9	39.1
Sept.	25.2	8.7	65.5	13.6	46.0	8.8	65.1	20.5	18.7	16.5	34.5	12.4	50.8	12.4	50.8	11.4	54.8

Table 14. Mean influent and filter effluent algal concentrations measured as volatile suspended solids for each sand size and hydraulic loading rate studied during Phase II under field conditions.

Month	Mean Monthly Influent Algae Conc. as Suspended Solids (mg/l)	Mean Monthly Effluent Volatile Suspended Solids Concentration and Percent Removals															
		Effective Size Sand, .17 mm										Effective Size Sand, .72 mm					
		Hydraulic Loading Rate - gpad										Hydraulic Loading Rate - gpad					
		400,000		500,000		600,000		700,000		800,000		400,000		500,000		600,000	
		mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.
May	2.2	2.2	-	4.5	-	1.6	27.3	-	-	-	-	3.8	-	1.5	31.8	3.5	-
June	3.6	1.6	55.6	2.4	33.3	1.3	63.9	-	-	-	-	1.9	47.2	1.7	52.8	2.2	38.9
July	23.6	4.5	80.9	6.8	71.2	4.4	81.4	9.8	58.5	5.6	76.3	5.5	76.7	4.9	79.2	6.5	72.5
Aug.	34.3	5.1	85.1	4.3	87.5	6.2	81.9	17.8	48.1	13.7	60.1	8.9	74.1	12.1	64.7	9.1	73.5
Sept.	22.3	2.7	87.9	5.6	74.9	2.5	88.8	6.6	70.4	8.4	62.3	4.8	78.5	2.1	90.6	4.1	81.6

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[2] G. K. Turner Associates, Palo Alto, California.

was not available to prewash the filters; therefore, it was necessary to wash with effluent. Much more material was washed from the 0.17 mm effective size sand because much of the fines were removed when preparing the 0.72 mm sand by screening.

At the 500,000 gpad hydraulic loading rate, monthly mean volatile suspended removals were essentially equal for the 0.17 and 0.72 mm effective size sands. Efficiencies fluctuated considerably from one sand to the other during the study period. But in general the 0.17 mm effective size sand produced a better quality effluent, particularly at the 600,000 gpad loading rate. Volatile suspended solids removal efficiencies appeared to be improving with the age of the filters, which is probably related to the washing of debris from the units (Table 14).

Examination of the effluent suspended solids concentrations at various hydraulic loading rates shown in Figure 4 indicates that the 0.72 mm filters were more efficient. However, the volatile suspended

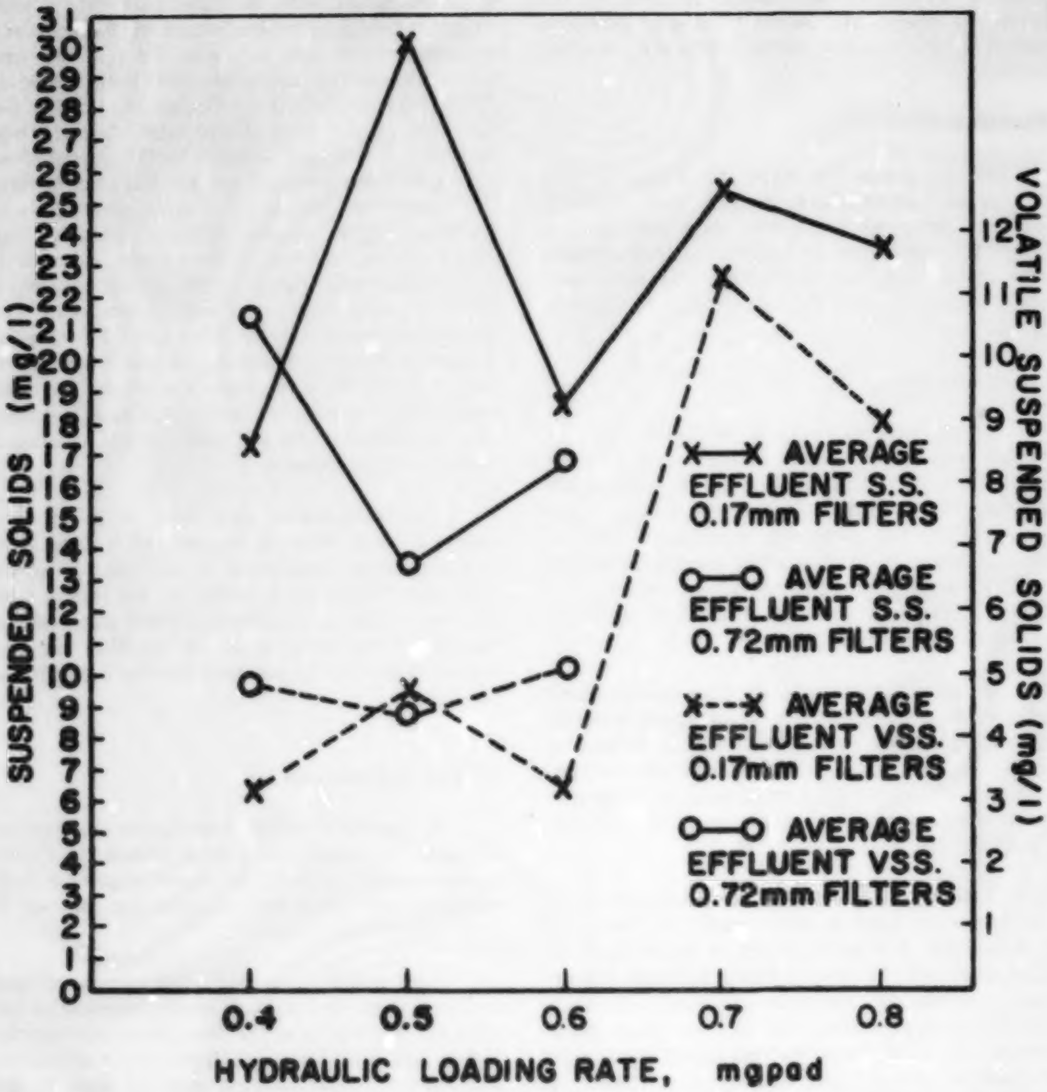

Figure 4. The relationship between the suspended and volatile suspended solids concentrations and the hydraulic loading rates for the field filters.

solids data show just the opposite. Again, this discrepancy is explained by the washing of silt and clay into the effluents.

Laboratory bioassay results indicated that as greater concentrations of algae were applied to the filters more viable cells passed through the 30 inches of sand. As shown in Table 14, during August when the algal concentration was at a maximum, more algae as volatile suspended solids passed the filters.

Although removal efficiencies appeared to improve with the age of the project, noticeable increases in removal efficiencies as the filters approached plugging did not occur. This is counter to the laboratory results and cannot be readily explained except by the variation normally occurring in solids analyses.

Bacterial removal

Stream standards recently adopted by the State of Utah include acceptable levels for both total and fecal coliform organisms. The standards require that a Class "C" water have an arithmetic monthly mean value of total and fecal coliform that does not exceed 5,000 and 2,000 per 100 ml, respectively (Middlebrooks et al., 1972).

Laboratory filters

Total coliform removals of better than 86 percent were obtained with all three sand sizes but due to the high applied counts (610,000 colonies/100 ml) the process was not able to meet the earlier noted standards in this particular application. Even at removals above 95 percent, lesser numbers of applied coliforms would have to be applied in order to meet Class "C" discharge standards. Calaway et al. (1952) presented similar results.

As the effective size of the sand was decreased, coliform removals increased, which agrees with the findings of Calaway et al. (1952). But, at the hydraulic loading rates employed, total coliform removals with the 0.72 mm (.0283 inch) sand were equal to those obtained in the filters containing 0.35 mm (.0137 inch) sand.

Total coliform removals were independent of the hydraulic loading rates employed, but it is doubted that this would apply at higher loadings. Calaway et al. (1952) found that at hydraulic loading rates approximately twice the rates used in this study that bacteria penetrated the bed to much greater depths. Therefore, more bacteria would be expected to pass the filter at higher loading rates. However, at the hydraulic loading rates of 100,000 (153.4 m^3/hectare-day), 200,000 (306.4 m^3/hectare-day), and 300,000 (459.5 m^3/hectare-day) gallons per acre per day, the coliforms removal was independent of loading.

Total plate counts for bacteria showed that the total number of bacteria in the filter influent was essentially unchanged by any of the three sand sizes studied. Also, hydraulic loading rate did not affect the numbers of bacteria present in the effluent.

Field filters

In an attempt to interpret the bacterial removal results obtained with the laboratory filters, total bacterial counts were performed on influent and effluent samples collected from the 0.17 mm and 0.72 mm effective size sands with both loaded at 500,000 gpad (765.90 m^3/hectare-day). After the 0.17 mm filters plugged and were cleaned, total bacterial counts were reduced by 99 percent three days after operation was resumed. But after 18 days of consecutive loading, the same filter effluent contained higher concentrations of bacteria than found in the influent. This increase in effluent concentration with time of operation after cleaning is probably attributable to two factors: (1) The bacterial population in the filter dies off during the drying period before removing the top few inches of sand, and (2) when operation is resumed, the clean sand serves as an efficient filter but as more and more bacteria penetrate the bed and multiply, more are washed into the effluent.

The intermittent sand filter is as much a biological as a physical process and is capable of producing large populations of bacteria within the filter bed. Treatment provided by the intermittent filter when used as a polishing device is accomplished throughout the entire depth of the filter and not limited to the top 12 inches of the bed as implied in other studies.

Effluent algal bioassays

As mentioned earlier, microscopic examination indicated that algae were passing through the filters. In an attempt to quantify the degree of passage, flask bioassays were employed to assess the number of viable algae in the effluents.

Algal growth, measured by an increase in light absorbancy, showed a much greater response in the effluents obtained from the filters when receiving the highest algal concentrations. Microscopic examination also showed higher concentrations of algae in the effluents when the highest concentration of algae was applied.

All of the flask assays exhibited a lag period of approximately three days before any significant growth occurred. This lag or acclimation period required for the algae to respond to a new environment could be advantageous in that it would allow the effluent to be transported considerable distances before an effect could develop. This would allow much of the algae that had passed the filter to settle out or be scavenged before growth could develop. If in the future it becomes necessary to meet more stringent requirements, disinfection would eliminate practically any surviving algal cells.

Microscopic examinations of the field filter effluents yielded similar results, but flask bioassays were not performed on the field filter effluents.

Filter conditions at plugging

Laboratory filters

Plugging did not occur during the three original loading periods used in the laboratory study. To obtain an estimate of the time required for plugging to occur, dosing was continued after Loading Period III without removing any sand from the beds and using algal suspensions from the model stabilization ponds. In order to estimate the plugging time under the most severe conditions evaluated, it was decided to continue loading at the Loading Period III concentrations.

A comparison of the effluent BOD_5 values at the time of plugging with those observed during normal operation showed no noticeable differences. Effluent suspended solids concentrations at the time of plugging were almost equal and near zero. This indicates that breakthrough does not occur in an intermittent sand filter. This finding is in agreement with the work of Ives (1961) which showed that as the specific deposit increased, the filter coefficient increased. Since the hydraulic head above the sand was not increased to the point that the filter coefficient was forced to decrease, the filters would plug when the filter coefficient was at a maximum. If it were practical to increase the head on intermittent sand filters, breakthrough might occur as in a high rate or pressurized filter.

Possibly the most important polishing mechanisms in intermittent sand filtration is the surface mat or "schmutzdecke" which is composed of suspended matter trapped on the surface of the filter. In this study the mat was composed primarily of algae that had been deposited upon the sand surface.

Following Loading Periods I and II, the filters did not seem to have a predominant surface skin of deposited suspended matter. The top 2 inches (5 cm) of sand seemed to be cemented together by the trapped suspended matter. Below this, the sand particles, although moist, were loose and apparently unaffected by suspended matter. At no time was any of the applied suspended matter detected at depths below the top 2 - 3 inches (5 - 7.5 cm). Sand 2 - 3 inches (5 - 7.5 cm) below the surface mat examined at the end of Loading Periods I and II did not appear to be affected by the applied algal suspensions. Once this sand had become dry, it was hard to tell it from new, clean sand. As the individual filters began to plug under the continued loadings following Loading Period III, a more predominant skin was noted on the sand surface. This skin was, in most cases, approximately one-sixteenth of an inch (1.6 mm) thick and covered the entire filter. During Loading Period III, the 0.17 mm (.0067 inch) and the 0.35 mm (.0137 inch) sands had surface mats that were moist, somewhat porous, and flat or well conformed to the sand surface. But the surface mat for the 0.72 mm (.0283 inch) sand, although moist, was curled and irregular. If a plugged filter is allowed to dry, the surface mat will curl away from the top surface of the sand. This indicates why raking or scraping has been shown to extend the length of filter runs.

Field filters

At the higher hydraulic loading rates employed in the field study, the surface mats for both the 0.17 and 0.72 mm sands followed essentially the same pattern as that observed in the laboratory. The 0.17 mm field filters operated approximately the same period of time before plugging as reported for the laboratory filters (Table 15). At the loading rates employed (400,000 to 600,000 gpad) with the 0.72 mm filters, plugging did not occur during the entire study. Based upon the results of both the laboratory and field studies, it appears that much higher hydraulic loading rates can be employed with the 0.72 mm filters. Higher hydraulic loading rates may result in more efficient solids removals with the 0.72 mm filters because of an increase in thickness of the mat that would accumulate on the surface and serve to trap more of the algae and debris. More detailed economic studies of the operation of the filters needs to be completed, but it appears that the hydraulic loading rate for the 0.17 mm effective size sand filters is limited to approximately 1 mgpad.

The 0.17 mm filters were cleaned by raking only which accounts for the relatively short periods of operation between the first and second plugging. If a conventional cleaning by removing the top 2 - 3 inches of sand had been performed, the second period of operation would have matched the initial period. However, because raking is an inexpensive method of extending the period between sand removals, it

Table 15. Operational history of the field filters during the second year.

Filter	Date Began Loading	Date 1st Plug	Ave. SS mg/l Applied	Type Cleaning	Date Loading Resumed	Date 2nd Plug	Ave. SS mg/l Applied	Date Loading Resumed	Type Cleaning	Plug 3rd Plug	Ave. SS mg/l Applied
A4	14 May	10 Aug	20.75	Rake	21 Aug	27 Sept[a]	27.57				
A5	14 May	10 Aug	20.75	Rake	21 Aug	27 Sept[a]	27.57				
A6	14 May	7 Aug	18.18	Rake	21 Aug	12 Sept	29.58	24 Sept	Rake	27 Sept[a]	24.57
A7	9 July	10 Aug	42.12	Rake	21 Aug	27 Sept[a]	27.57				
A8	9 July	10 Aug	42.12	Rake	15 Aug	7 Sept	34.99	7 Sept	Rake	27 Sept[a]	23.82
C4	14 May	27 Sept[a]	25.10								
C5	14 May	27 Sept[a]	25.10								
C6	14 May	27 Sept[a]	25.10								

[a] Project ends.

should be considered part of the routine operating procedure.

Time of operation

Laboratory filters

Figure 5 shows the effect of sand size and run time before plugging occurs. It is again evident that the finer sands produce the lowest effluent suspended solids concentrations. But it is also quite apparent that this improved effectiveness was attained at the expense of a reduction in operation time before plugging.

As the operating time increased, an increase in the suspended solids removal efficiency was noted. This is the same as noted by Ives (1961); i.e., as the specific deposit increases, the filter coefficient increases. Knowledge of this situation could prove to be valuable when operating a number of filters. Regular analysis of the effluent for suspended solids would allow one to predict when plugging was likely to occur.

Continued operation eventually caused plugging in all filters. The results show that the 0.72 mm (.0283 inch) filter operated 175 consecutive days before plugging when loaded at a mean algal concentration of 51 mg/l, the 0.17 mm (.0067 inch) sand operated 68 consecutive days at a mean algal concentration of 43 mg/l, and the 0.35 mm (.0137 inch) sand operated 99 consecutive days at an applied algal concentration of 45 mg/l.

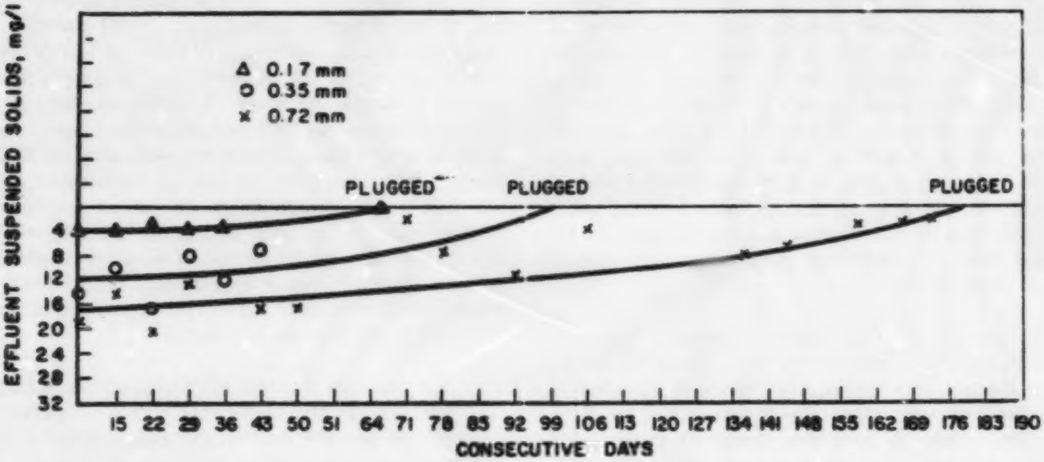

Figure 5. Observed times of operation under approximately 45 mg/l applied algae afforded by each sand size under laboratory conditions and the resulting effect on effluent suspended solids concentration. (Loading Period III plus continuation.)

Table 16 shows in more detail the operational results for all the filters during the continuation of Loading Period III. Removing the top 4 inches (10 cm) of sand from the filters after plugging, replacing it with new sand, and putting the unit back in operation gives second performance periods generally less than the original period. Longer operating periods were expected with the lower hydraulic loading rates; however, the 0.17 mm (.0067 inch) and 0.35 mm (.0137 inch) effective size filters at the lowest loading rate were the first to plug.

Figure 6 shows that the highest hydraulic loading rate studied also allowed greater volumes of applied water to pass the filter bed before plugging occurred. As the figure shows, the result was the same for all the sand sizes studied.

Field filters

As reported for the laboratory filters, the finer sand produced a superior effluent in all categories measured, and again this higher efficiency was attained at the expense of a reduction in operation time before plugging (Table 15).

The increase in suspended solids removal efficiency with increasing operating time was observed for the field filters, but the effluent concentrations appeared to reach a limit and did not continue to drop until plugging occurred. The removal efficiency increase with time of operation in the field filters appeared to be more closely associated with the washing of fines from the filters. However, there is no reason not to expect similar performances between the laboratory and field filters, and it may be that the lack of a decrease in effluent solids concentration is attributable to a continuous washing of fines from the filters up to plugging. After more than one summer of operation, it is likely that a pattern as observed in the laboratory would evolve in the field. Since the 0.72 mm field filters did not plug during the summer of operation, it is possible that the laboratory study results would have been duplicated had the project continued, or had the suspended solids concentrations in the influent been increased.

The 0.72 mm filters operated 137 consecutive days without plugging when loaded at 0.4, 0.5, and 0.6 mgpad with a lagoon effluent containing an average suspended solids concentration of 25 mg/l. It was not surprising that these units did not plug, because laboratory units with the same sand and a hydraulic loading rate of 0.3 mgpad operated 175 days before plugging and were dosed with a lagoon effluent containing an average suspended solids concentration of 51 mg/l.

The consecutive days of operation for the 0.17 mm filters appear to be directly related to the hydraulic loading rates. Figure 7 shows that up to a loading rate of 0.6 mgpad the filters operated approximately 100 days before plugging when receiving a lagoon effluent containing a mean algal concentration of 20 mg/l. During these 100 days, the filter influent algae concentration ranged from 4 to 51 mg/l. At loading rates of 0.7 and 0.8 mgpad, the 0.17 mm filters operated only 32 consecutive days when receiving a lagoon effluent containing a mean suspended solids concentration of 42 mg/l, and a range of concentrations varying between 30 and 50 mg/l. Because of the large difference in the mean applied suspended solids concentrations, it is impossible to compare the performances at the two

Table 16. Results of the continuation of Loading Period III showing approximate period of operation by the laboratory filters using two cleaning methods. Loading Period III began 11/1/72 at which time all filters had the top 4 inches (10 cm) of sand removed and replaced with new sand.

Filter	Date First Plugging	Mean Applied SS to Date	Type Cleaning	Date Put Back in Use	Date Second Plugging	Mean Applied SS from First Plugging	Type Cleaning	Date Put Back in Use	Date Third Plugging
SF 11	4/14	51.46	raking	4/20/73					
SF 12	4/27	51.08							
SF 13	4/25	51.08							
SF 21	12/26/72	44.57	scraping	1/10/73	2/6/73	51.76	scraping	2/8/73	
SF 22	12/27/72	44.57	scraping	1/10/73	4/2/73		raking		
SF 23	3/5/73	48.79	raking	3/8/73					
SF 31	12/18/72	46.35	scraping	1/10/73	1/20/73	53.59	scraping	1/30/73	
SF 32	1/28/73	45.19	scraping	1/30/73					
SF 33	1/17/73	44.47	scraping	1/30/73	3/14/73	63.56	raking	3/21/73	4/3/73

hydraulic loading rates, or to develop relationships between consecutive days of operation and the hydraulic loading rate.

The results are useful in estimating the number of time during an algae growing season that the filters must be raked and cleaned. During the early spring and summer it is likely that the units will perform effectively for the first 3 months, and removing the top 2 to 4 inches of sand the units should perform a minimum of one month even at very high algal concentrations in the filter influent. It is possible that the consecutive days of operation at the 0.7 and 0.8 mgpad hydraulic loading rates will match those at the 0.4 to 0.6 mgpad rates when receiving equal concentrations of influent algae. Length of operation and the economics of maintenance will be answered in the continuation of the project which will be conducted on a prototype scale.

When the filters plugged, the surface mat and approximately the top 2 inches of sand were raked and broken up and then placed in service again. Figure 7 shows that there was not a relationship between hydraulic loading rate and consecutive days of operation following the raking. The 0.17 mm filters receiving 0.4, 0.5, and 0.7 mgpad of lagoon effluent had loaded for 38 days and were still operating after the first raking when the project was terminated. The filters receiving 0.6 and 0.8 mgpad plugged within 22 days after the raking. Mean suspended solids concentrations in the lagoon effluent applied to all of the 0.17 mm filters following raking were approximately equal, but the two filters that plugged the second time did receive the highest concentrations of algae, 29.6 mg/l for the 0.6 mgpad loading rate and 35.0 mg/l for the 0.8 mgpad loading rate.

Although direct comparisons of the lengths of performance at the various hydraulic loading rates are difficult, it is obvious that the length of runs for all of the sands and hydraulic loading rates are of adequate length to make intermittent sand filtration competitive with all other processes available to upgrade lagoon effluents to meet new water quality standards.

Figure 8 shows the volume of lagoon effluent applied to the field filters during the 137 days of operation. As reported for the laboratory filters, the

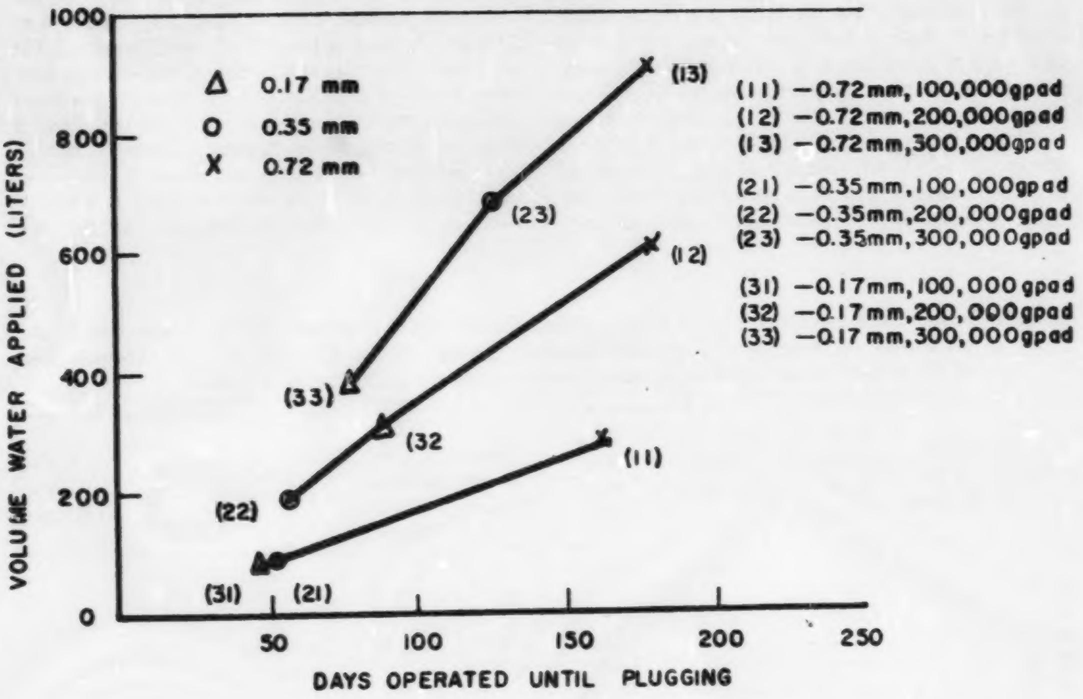

Figure 6. The relationship observed between the volume of water applied until plugging occurred under laboratory conditions.

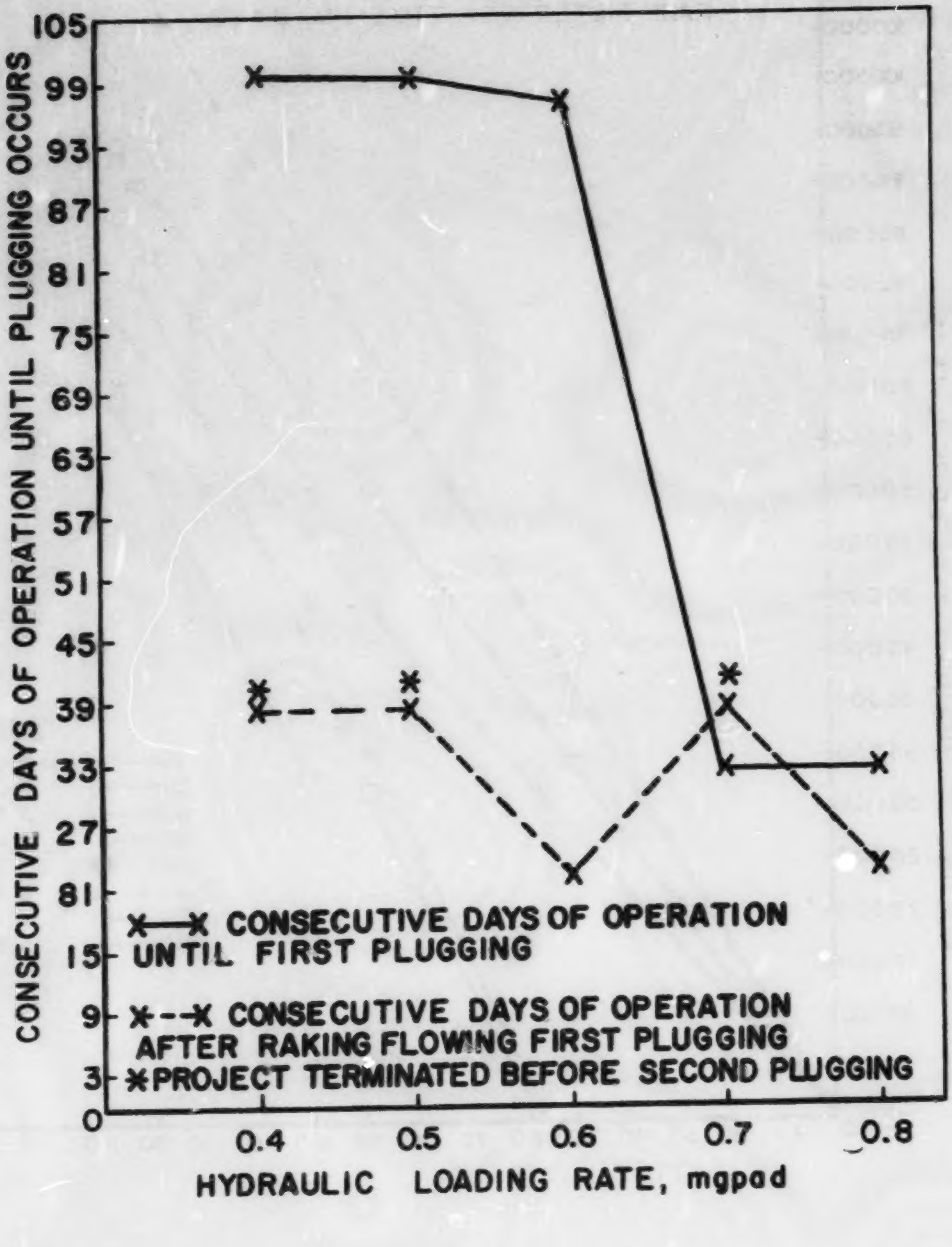

Figure 7. Consecutive days of operation until plugging occurred in the 0.17 mm effective size sand filters at various hydraulic loading rates.

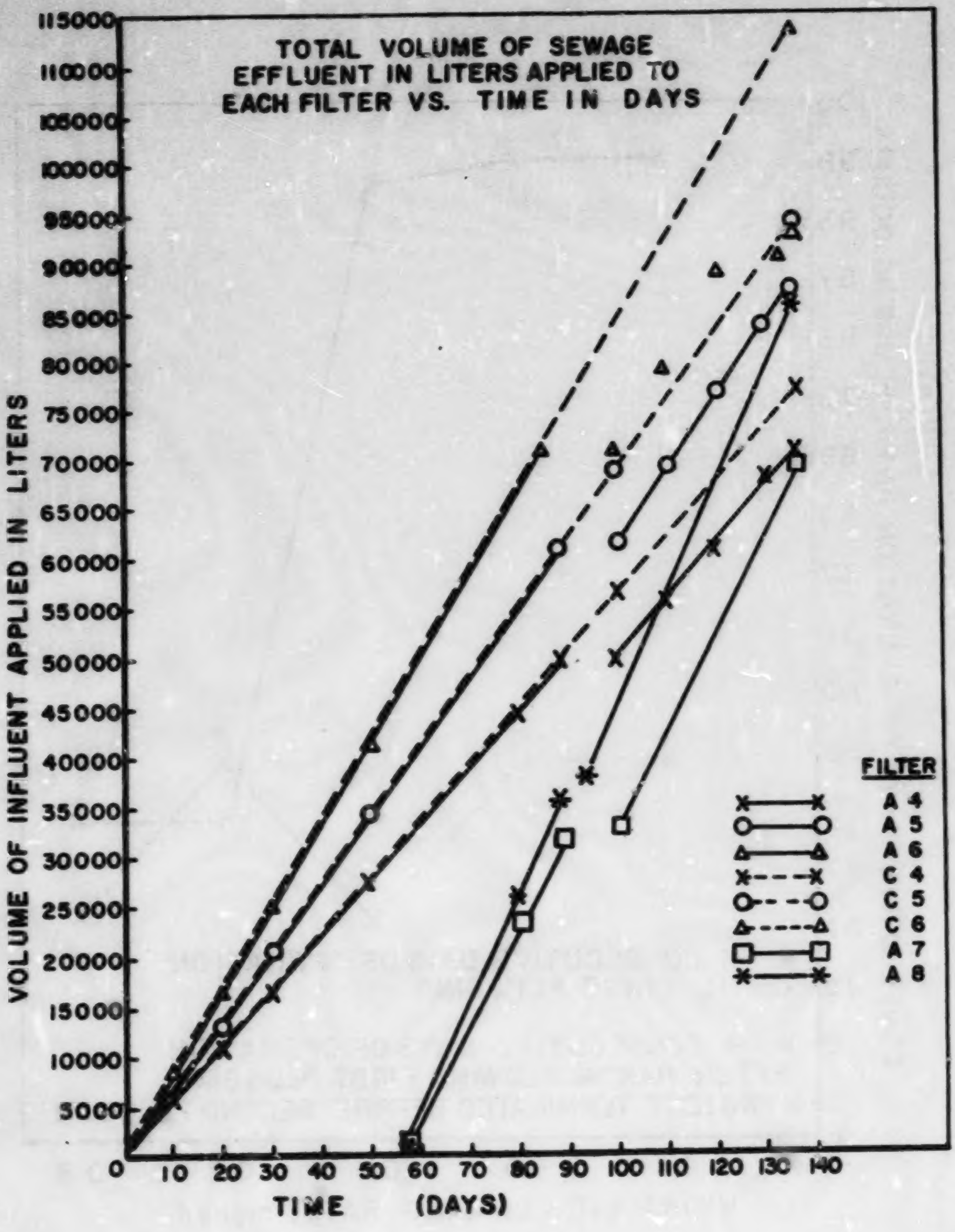

Figure 8. The relationship observed between the volume of water applied until plugging occurred under field conditions. Discontinuity in the lines represents the rest and cleaning period following plugging.

greatest volume of water was treated in a given time span by the filters receiving the highest hydraulic loading rates even when plugging occurred and it was necessary to rest the filter and rake the surface before returning it to operation.

Overall evaluation of the process

Ability to meet present state standards

Intermittent sand filtration was evaluated to assess its capability to produce an effluent that would meet the Utah Class "C" stream standards shown in Table 17 when imposed as discharge standards. In a system such as the Logan City wastewater stabilization ponds, the intermittent sand filter would produce an effluent meeting Class "C" discharge standards 99 percent of the time. The 0.17 mm (.0067 inch) effective size laboratory filters only produced an effluent with a mean BOD_5 greater than 5 mg/l (maximum effluent BOD_5 equal 8 mg/l) when loaded at the highest algal concentration and with an influent BOD_5 concentration averaging 36 mg/l. The BOD_5 concentration (36 mg/l) in the influent during Loading Period III was much higher than normally obtained from a well operated secondary wastewater treatment plant. The 0.17 mm field filters produced an effluent with a BOD_5 concentration of less than 5 mg/l on all days of operation when loaded at 0.6 mgpad or less. Effluent BOD_5 concentrations for the filter loaded at 0.7 mgpad exceeded 5 mg/l on only 2 days out of 69 days of operation and the maximum value in the effluent was 6.7 mg/l. The average BOD_5 in filter A7 effluent was 3.7 mg/l for the entire period of 69 days. A loading of 0.8 mgpad produced an effluent of slightly poorer quality but still reduced the BOD_5 to a mean value of 4.1 mg/l. Thus, most properly operated secondary treatment plants in the state would be able to meet Class "C" discharge standards with the addition of intermittent sand filtration. Even under such heavy BOD_5 loadings as studied during Loading Period III in the laboratory, reductions were greater than 80 percent for the 0.17 mm (.0067 inch) sand at all hydraulic loading rates.

Middlebrooks et al. (1972) reported the effluent characteristics of 11 existing wastewater treatment plants in the State of Utah, some of which were heavily overloaded. Seven were trickling filters and five were wastewater stabilization ponds. Assuming that equivalent reductions in BOD_5, suspended solids, and coliform organisms would be obtained by intermittent sand filtration on all types of secondary treatment plant effluents, seven of the eleven plants would be able to meet the BOD_5 standards for Class "C" discharged waters by adding intermittent sand filters. If the overloading were corrected and the plants operated properly, all 11 plants could meet Class "C" discharge standards by installing intermittent filters. Several of these plants were serving metropolitan areas, and it may not be feasible to utilize intermittent filters because of land limitations and economic constraints usually associated with metropolitan areas.

On the basis of the mean total coliforms per 100 ml reported, five of the eleven Utah wastewater treatment facilities would be able to meet Class "C" discharge standards by the addition of intermittent sand filtration. If the plants were not overloaded, in all probability the coliform requirements could be met in all 11 plants. Again, the addition of a disinfection step would aid materially in meeting coliform removal requirements as well as eliminate the contributions to a downstream algae bloom problem.

Class "C" discharge requirements for pH value and dissolved oxygen are normally easily met by secondary treatment, and the intermittent sand filtration of these effluents further refines effluents. The pH values of the Logan lagoon effluents were approximately equal to a value of 9. When passed through the filters, the pH was reduced to values approximately within the limits imposed. Six to nine mg/l of dissolved oxygen were readily produced by intermittent sand filtration which also meets Class "C" water standards.

Cost estimate

A general approach was taken in the preparation of the cost estimates for an effluent polishing intermittent sand filter process. The estimates shown

Table 17. Class "C" stream standards for the State of Utah (Middlebrooks et al., 1972).

Parameter	Concentration or Unit
pH	6.5 - 8.5
Total Coliform, Monthly Arithmetic Mean	5,000/100 ml
Fecal Coliform, Monthly Arithmetic Mean	2,000/100 ml
BOD_5, Monthly Arithmetic Mean	5 mg/l
Dissolved Oxygen	> 5.5 mg/l
Chemical and Radiological	PHS Drinking Water Standards

for initial plant construction outlays are of a higher degree of reliability than the values estimated for operation. Much better estimates of operational expenses will be afforded by the future prototype study.

The in-place total construction cost estimates were prepared through the aid of a local consulting engineering firm. Thus, they are representative of the outlay necessary to construct a typical intermittent sand filter process in the intermountain area during 1973.

The construction and annual operation cost estimate shown for the 15 mgd Logan City facility is not as general in nature as the other estimates. This estimate was based upon two assumptions: One, the process would be located such that pumping of the applied effluent was not necessary; two, additional cost for land is not necessary as the final one and a half existing tertiary ponds would be drained and the polishing filters would be located within these boundaries. Also, the 15 mgd Logan stabilization pond system is presently the largest existing facility of this type in Utah. A cost estimate for this facility will then provide an expense evaluation for the entire range of stabilization pond systems in Utah.

A large difference was found between locally available filtering media and specially prepared media, so an economic evaluation of the two types of media was made. In this case, the 0.17 mm (.0067 inch) size media was locally available and the 0.35 mm (.0137 inch) and the 0.72 mm (.0283 inch) sizes were specially prepared. The specially prepared media in this area was found to be more than five times more costly than the locally available media. Based on the assumptions that the 0.17 mm (.0067 inch) locally available media was approximately two and one-half times more costly to operate than the 0.72 mm (.0283 inch) media, the 0.17 mm (.0067 inch) media was found to be the economic choice for a 1 mgd and the 15 mgd existing lagoon system.

The construction costs determined in Estimates 1 through 4, Table 18, reflect a paired bed operation designed at 300,000 gpad (459.5 m^3/hectare-day) and 800,000 gpad (1225.4 m^3/hectare-day) and the application of the effluent to the filter in less than 90 minutes. It was assumed that in a municipal construction effort such as this, at least 75 percent of the construction cost would be funded by federal aid. Also, costs without federal assistance are reported.

The construction costs for Estimate 5, Table 18, reflect an optimum design situation for a 1 mgd facility. Conditions considered optimum are minimum bed area operated under scheduled rotation, no pumping required for dosing, locally available media, and plastic bed liners not required. Under these conditions with the aid of federal funds, a filter process designed at 800,000 gpad (1225.4 m^3/hectare-day) for a 1 mgd facility would cost the community $14,500 to construct.

Sand or media expense is approximately 25 percent of the total construction cost. Also, the plastic liner for the bed is approximately 25 percent of the total construction cost. Whether or not the liners are a required expense in constructing effluent polishing intermittent sand filters will depend on the specific conditions and regulations governing each location and installation of this process. As shown in Estimate 5, considerable savings are made by not installing the plastic bed liners. In rural areas, land costs for this process are less than 5 percent of the total construction costs.

Costs per million gallons of effluent produced are shown in Table 18 with and without federal assistance. Without federal funds, the costs are greatly increased. The effect of an optimum condition application is noted by the cost of $16 per million gallons (Table 18). Combined effects of larger scale operation and specific application, which in this case held conditions near optimum, are noted by the $15 per million gallons cost for the Logan City facility.

Table 18. Estimated cost per million gallons of filtrate produced by various designs of an effluent polishing intermittent sand filter process.

Application conditions	Design Flow Rate	Design Hydraulic Loading Rate	Effective Sand Size	Cost With Federal Assistance $/10^{3}gallons	Cost Without Federal Assistance $/10^3 gallons
General (Estimate 1)	1 mgd	0.3 mgad	.17 mm	$47	$115
General (Estimate 2)	1 mgd	0.8 mgad	.17 mm	$33	$ 61
General (Estimate 3)	1 mgd	0.8 mgad	.72 mm	$46	$145
Specific (Estimate 4)	15 mgd	0.6 mgad	.17 mm	$15	$ 48
Optimum (Estimate 5)	1 mgd	0.8 mgad	.17 mm	$16	$ 26

Finally, for the general applications estimates when a 1 mgd plant constructed with 0.17 mm (.0067 inch) effective size locally available media is compared to one constructed of a specially prepared media. The cost of operation and media using the 0.17 mm (.0067 inch) effective size sand designed for a hydraulic loading rate of 0.3 mgd is essentially equal to the operation and media costs for the coarser 0.72 mm (.0283 inch) effective size specially produced sand. If the 0.72 mm (.0283 inch) effective size specially prepared sand filter were designed using much higher loading rates and optimum conditions, the cost per million gallons for this particular sand would decrease to the point where it would become economically competitive.

From the present understanding of the operation of effluent polishing intermittent sand filters, a cost ranging between $15 to $47 per million gallons can be assumed to be representative of this process. Table 19 lists alternative methods to meet Class "C" water standards and their estimated costs as reported by Middlebrooks et al. (1972). Based on these values, the earlier stated cost for an effluent polishing intermittent sand filter process is quite competitive. There are many avenues of approach that may be taken to produce the same high quality effluent of this process at even lower expense. Coupling this possibility with the fact that a majority of the existing wastewater effluents in Utah can be upgraded to meet Class "C" water standards by the addition of this process, intermittent sand filtration of wastewater effluents has been found to be an economically feasible method of wastewater effluent polishing.

Summary and Conclusions

The major objective of this study was to evaluate the performance of the intermittent sand filter and to determine if it was capable of upgrading existing wastewater treatment plants in the State of Utah to meet Class "C" water quality standards.

Table 19. Cost of alternative methods of polishing wastewater effluents (Middlebrooks et al., 1972).

Method	Cost per 10^6 gal.
Chemical treatment (solids contact)	$60-130
Granular or mixed media filtration w/chem	$50
Dissolved air flotation	$110
Electrodialysis	$200
Microstraining	$18

Hydraulic loading rate was found to have little effect on any of the parameters studied in the laboratory portion of the study. In the field experiments at much higher hydraulic loading rates and varying algal concentrations, suspended and volatile suspended solids removal appeared to decrease with an increase in hydraulic loading. Although significant quantities of applied algae were removed by filtration, cells were found to pass the entire bed depth. Sand size was found to have a general effect on the quality of the effluent produced by filtration. Sand size was also found to be related to the time of operation before plugging occurred. It was concluded that intermittent sand filtration was capable of upgrading a majority of the existing wastewater effluents in Utah to meet Class "C" water standards.

In addition to the above, the following general observations and conclusions were made:

1. Smaller effective size sands better oxidize nitrogen compounds.

2. Hydraulic loading rate has little effect on ability of the sand filter to oxidize nitrogen at the loading rates studied in the laboratory.

3. The nitrogen form which is being oxidized is principally that of ammonia.

4. After establishing equilibrium with the media, intermittent sand filters do not remove a significant quantity of dissolved phosphorus compounds.

5. Hydraulic loading rate has little effect on BOD_5 removal when secondary wastewater effluent is applied to intermittent sand filters with bed depths of 30 inches.

6. BOD removal increased as the effective size of the sand decreased. The 0.17 mm effective size sand filters produced a project low mean effluent BOD_5 concentration of 1.6 mg/l at the 0.4 mgpad loading rate and a high value of 4.1 mg/l at 0.8 mgpad. The project mean effluent BOD_5 concentration for the 0.72 mm effective size sand filters ranged from 5.0 to 5.5 mg/l for the 0.4, 0.5, and 0.6 mgpad hydraulic loading rates.

7. BOD_5 removal was independent of the applied BOD value at the concentrations studied in the laboratory.

8. Viable algal cells passed the entire depth of all the filter sands studied.

9. Hydraulic loading rate did not affect the algae or suspended solids removal efficiency at the

100,000 (153.4 m^3/hectare-day), or 200,000 (153.4 m^3/hectare-day), or 300,000 (454.9 m^3/hectare-day) gallons per acre-day loadings employed in the laboratory study. The effects of hydraulic loading rate on SS removals in the field studies were inconclusive because of the large quantities of fines washed from the filters, but volatile suspended solids removals did indicate a reduction in removal efficiency as the hydraulic loading rate was increased.

10. Smaller effective size sands produced better algal or suspended and volatile suspended solids removals.

11. Sand size was not a significant factor in algal removal at applied algal concentrations of 15 and 30 mg/l, but was significant when the concentration was increased to 45-50 mg/l in both the laboratory and field filters.

12. Intermittent sand filtration produced a 90 percent reduction in the total coliform count in the laboratory filters.

13. Coliform removal was independent of the hydraulic loading rates employed in the laboratory filters.

14. Total bacterial counts as measured by the standard plate count apparently was not reduced by any of the sands studied.

15. Filter plugging causes no decline or improvement in the effluent BOD at or near the time of plugging.

16. Immediately before plugging occurred in the laboratory filter, the filter effluent suspended solids concentrations were approximately zero. As the filter operates with time, the suspended solids removal efficiency increases reaching a maximum point at the time of plugging. This did not occur in the field, but if fines were washed from the filter before placing it in operation, it is likely that a similar pattern would occur.

17. At hydraulic loading rates of 0.4 to 0.6 mgpad the 0.17 mm effective size sand filters will operate approximately 100 days before cleaning is required when receiving a lagoon effluent containing a mean suspended solids concentration of 20 mg/l.

18. At loading rates of 0.7 and 0.8 mgpad the 0.17 mm filters will operate 32 consecutive days before requiring cleaning when receiving lagoon effluent containing a mean suspended solids concentration of 42 mg/l.

19. Laboratory filters containing sands of 0.72 mm effective size operated 175 consecutive days before plugging when dosed with a lagoon effluent containing a mean suspended solids concentration of 51 mg/l at a rate of 0.3 mgpad.

20. Field filters containing 0.72 mm effective size sand operated 137 consecutive days before terminating the study without plugging when loaded at 0.4, 0.5, and 0.6 mgpad with a lagoon effluent containing a mean suspended solids concentration of 25 mg/l.

21. If operated and loaded properly, all existing wastewater treatment plants in the State of Utah could be upgraded by intermittent sand filtration to meet Class "C" state standards.

22. Based upon current cost figures it appears that an effluent polishing intermittent sand filter process can be constructed and operated for a cost ranging between $15 to $47 per million gallons of filtrate.

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INTERMITTENT SAND FILTRATION TO UPGRADE LAGOON EFFLUENTS–PRELIMINARY REPORT

J. H. Reynolds, S. E. Harris, D. Hill,
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Introduction

Nature of the problem

Approximately 90 percent of the wastewater lagoons in the United States are located in small communities of 5,000 people or less. These communities, many with an average daily wastewater flow rate of only 175,000-200,000 gallons, do not have the resources to keep men at the lagoon sites throughout the day. A high degree of technical knowledge is usually lacking in these communities. Often only periodic inspection or maintenance is carried out by the general municipal employees. Therefore, the development of a relatively inexpensive, low operation and maintenance method for polishing lagoon effluent to meet the requirements of PL 92-500 is needed.

There are many sections of the country that are still fortunate to be surrounded by large areas of open and relatively inexpensive land. It was originally due to this reason that many of these communities adopted waste stabilization lagoons as a means of wastewater treatment. Although this treatment scheme requires large tracts of land, the important consideration was that it gave a satisfactory effluent for minimum cost and maintenance. But now a better quality effluent is necessary. If small cities and towns are to economically produce a higher quality effluent, some form of treatment must be utilized that will continue to take advantage of the large areas of relatively inexpensive land surrounding these communities. One method of treatment that capitalizes on the availability of large land areas is intermittent sand filtration.

In most areas of the country where intermittent sand filtration has been used, the lack of large inexpensive tracts of land was a major factor contributing to a decline in use. Thus, the relatively inexpensive tracts of rural land available near many lagoon sites is a definite asset. Intermittent sand filtration becomes even more economically attractive if filter media are available locally.

Objectives

The objective of this study was to evaluate and compare the performance of a full-scale intermittent sand filter with results obtained using laboratory and pilot field scale intermittent sand filters as a polishing process that would upgrade existing wastewater treatment facilities. Particular attention was directed toward ascertaining the effectiveness of the intermittent sand filter as a means of removing the highly variable quantities of algae present in stabilization ponds during the warmer months of the year. The ultimate objective is to develop design criteria for intermittent sand filters that will consistently produce an effluent of a quality that would meet stringent water quality standards.

Previous Investigations

Historical background

Use of intermittent sewage filtration began in this country in the late nineteenth century. The first intermittent sewage filters were put in use in 1889 in Massachusetts. For many years their use was centered in the New England area. By 1945, approximately 450 intermittent filter plants were in operation in this country. But later reports showed a decrease to 398 in use by 1957. It was also noted (ASCE and FSIWA, 1959) that 94 percent of those still in use by 1957 were serving communities with populations under 10,000.

The intermittent sewage filter has long been known to have the ability to produce effluents of relatively high quality as did the slow sand filter for culinary waters. The decline of intermittent sewage filters was related to the same factors that caused the decline of slow sand filters—an increase in quantity of

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water to be filtered due to a growing population, and to the rising costs of land.

Intermittent sand filtration, as noted earlier, began in the New England area of this country. Located in Lawrence, Massachusetts, was the Lawrence Experiment Station at which many of the first studies on intermittent sand filtration were accomplished. This region of the country was ideal for the application of such a process as intermittent sand infiltration. Many small rural communities were developing to the point that it was necessary to treat their wastewater at a central plant which was economical for the small town. Land to build the filters upon was readily available at reasonable rates and there was also abundant quantities of well graded bank run sand available. These conditions encouraged efforts in research at the Lawrence Experiment Station to improve the intermittent sand filter. As a result of this experimentation and success, the use of intermittent sand filters increased.

Following World War II, many people found the mild climate and sparsely populated land of Florida an ideal place to live following retirement. Large numbers of small residential centers such as isolated tourist courts, motels, trailer parks, drive-in theaters, consolidated schools, and housing developments began to spring up all over Florida. It was soon realized that an economic method of sewage treatment would be necessary for these small communities. Thus, the study of intermittent sand filtration was undertaken at the University of Florida at Gainesville, (Calaway et al., 1952; Furman et al., 1949; and Grantham et al., 1949). Much of the modern day knowledge on intermittent sand filters has come out of the studies carried out at Gainesville. All of this work was concentrated on treating raw or primary effluent wastewaters. A detailed review of the University of Florida study and other investigations dating back to 1908 has been presented by Marshall and Middlebrooks (1974).

Laboratory and pilot scale studies

Marshall and Middlebrooks (1974) evaluated the performance of laboratory and pilot scale intermittent sand filters to determine if the process was capable of upgrading existing wastewater treatment plants in the State of Utah to meet Class "C" water quality standards (BOD $\leqq$ 5 mg/l, pH = 6.5-8.5, total coliform $\leqq$ 5,000/100 ml, fecal coliform $\leqq$ 2,000/100 ml, and D.O. $>$ 5.5 mg/l). Effective size of the sand, hydraulic loading rate, and algal concentrations were the variables studied. Hydraulic loading rate was found to have little effect on any of the parameters studied in the laboratory portion of the study. In the field experiments at much higher hydraulic loading rates and varying algal concentrations, suspended and volatile suspended solids removal appeared to decrease with an increase in hydraulic loading. Although significant quantities of applied algae were removed by filtration, cells were found to pass the entire bed depth. Sand size was found to have a general effect on the quality of the effluent produced by filtration. Sand size was also found to be related to the time of operation before plugging occurred. BOD removal increased as the effective size of the sand decreased. The 0.17 mm effective size sand filters produced a project low mean effluent BOD_5 concentration of 1.6 mg/l at the 0.14 mgad loading rate and a high value of 4.1 mg/l at 0.8 mgad. The project mean effluent BOD_5 concentration for the 0.72 mm effective size sand filters ranged from 5.0 to 5.5 mg/l for the 0.4, 0.5, and 0.6 mgad hydraulic loading rates. Hydraulic loading rate did not affect the algae or suspended solids removal efficiency at the 100,000 (153.4 m^3/hectare-day), or 200,000 (306.8 m^3/hectare-day), or 300,000 (454.9 m^3/hectare-day) gallons per acre-day loadings employed in the laboratory study. The effects of hydraulic loading rate on SS removals in the field studies were inconclusive because of the large quantities of fines washed from the filters, but volatile suspended solids removals did indicate a reduction in removal efficiency as the hydraulic loading rate was increased. Immediately before plugging occurred in the laboratory filter, the filter effluent suspended solids concentrations were approximately zero. As the filter operated with time, the suspended solids removal efficiency increased reaching a maximum point at the time of plugging. This did not occur in the field, but if fines were washed from the filter before placing it in operation, it is likely that a similar pattern would occur. At hydraulic loading rates of 0.4 to 0.6 mgad the 0.17 mm effective size sand filters were found to operate approximately 100 days before cleaning is required when receiving a lagoon effluent containing a mean suspended solids concentration of 20 mg/l. At loading rates of 0.7 and 0.8 mgad the 0.17 mm filters will operate 32 consecutive days before requiring cleaning when receiving lagoon effluent containing a mean suspended solids concentration of 42 mg/l. Based upon current cost figures it appears that an effluent polishing intermittent sand filter process can be constructed and operated for a cost ranging between \$15 to \$47 per million gallons of filtrate. It was concluded that intermittent sand filtration was capable of upgrading a majority of the existing wastewater effluents in Utah to meet Class "C" water standards.

Experimental Procedures

Experimental facility

The experimental facility is located at the Logan Municipal Sewage Lagoons as shown in Figure 1. The facility consists of six, 25 ft x 36 ft (900 sq ft

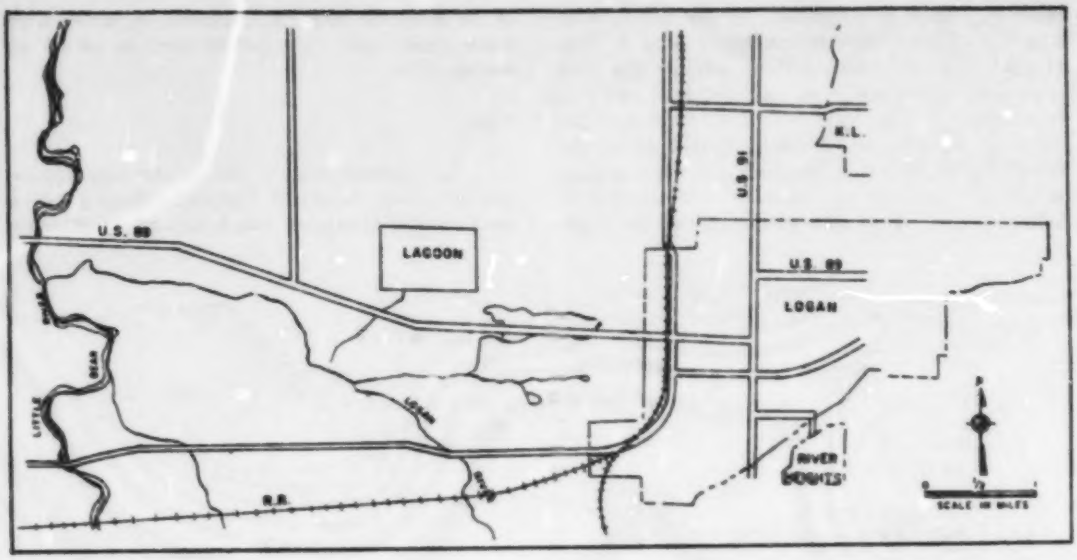

Figure 1. Location map.

surface) intermittent sand filters. A cross section of a typical filter is shown in Figure 2. The soil embankment around each filter is constructed of bank run granular fill material. Each filter contains 1 foot of graded gravel (1/4 in max. diameter to 1 1/2 in max. diameter) and 3 feet of pit run concrete sand. The sand has an effective size of 0.170 mm and a uniformity coefficient of 9.74. A sieve analysis of the sand is shown in Table 1. The filters are lined with a vinyl liner (10 mm) to prevent infiltration of subsurface groundwater or exfiltration of filter influent. The filter construction provides 3 feet of freeboard above the sand surface.

The filters are loaded once daily with either tertiary or secondary effluent from the Logan Municipal Sewage Lagoon system (depending on which effluent has the highest suspended solids concentration). The hydraulic loading of each filter is accomplished in less than 30 minutes. The loading rates being studied are 0.2, 0.4, 0.6, 0.8, 1.0, and 1.2 mgad. When the total amount of applied influent to a filter does not drain through the filter in 24 hours, the filter is considered to be plugged and is taken out of service and cleaned.

Table 1. Sieve analysis of filter sand.[a]

U.S. Sieve Designation Number	Size of Opening (mm)	Percent Passing (%)
3/8"	3/8"	100.0
4	4.76	92.1
10	-	61.7
40	0.42	27.0
100	0.149	6.2
200	0.074	1.7

[a] e = 0.170 mm; u = 9.74.

Cleaning is accomplished by removing the top 1/2 to 2 inches of filter sand. The sand is stockpiled and will eventually be washed. The organic washing from the sand will be recycled to the primary lagoon and the clean filter sand will be replaced in the filter.

Sampling and analyses

Samples of filter influent and effluent are collected and analyzed for suspended solids and volatile suspended solids three times a week, biochemical oxygen demand (BOD_5) once a week and chemical oxygen demand (COD), total phosphorus, orthophosphorus ammonia, nitrite, pH, temperature, and dissolved oxygen are monitored once each week. The effluent samples are collected 2 hours after each filter is loaded. The procedure employed for each analysis is shown in Table 2.

Results and Discussion

General

Data collection began on July 12, 1974, after the filters had been "washed" to remove dirt and fine inorganic material generated by the filter construction. Data collection continued until a filter plugged and was taken out of service. The data presented in this report were collected during the first experiment period (July 12, 1974, to August 22, 1974) of the project. All data collected during the report period have been averaged and these average values are reported in Table 3. The number of individual data points averaged depends on the length of the particular filter run. However, in no case were fewer than three data points used to obtain an average value.

Algae

The characteristics of the influent algal population are shown in Table 4. The predominant algae are small coccoid blue-green, which have been tentatively

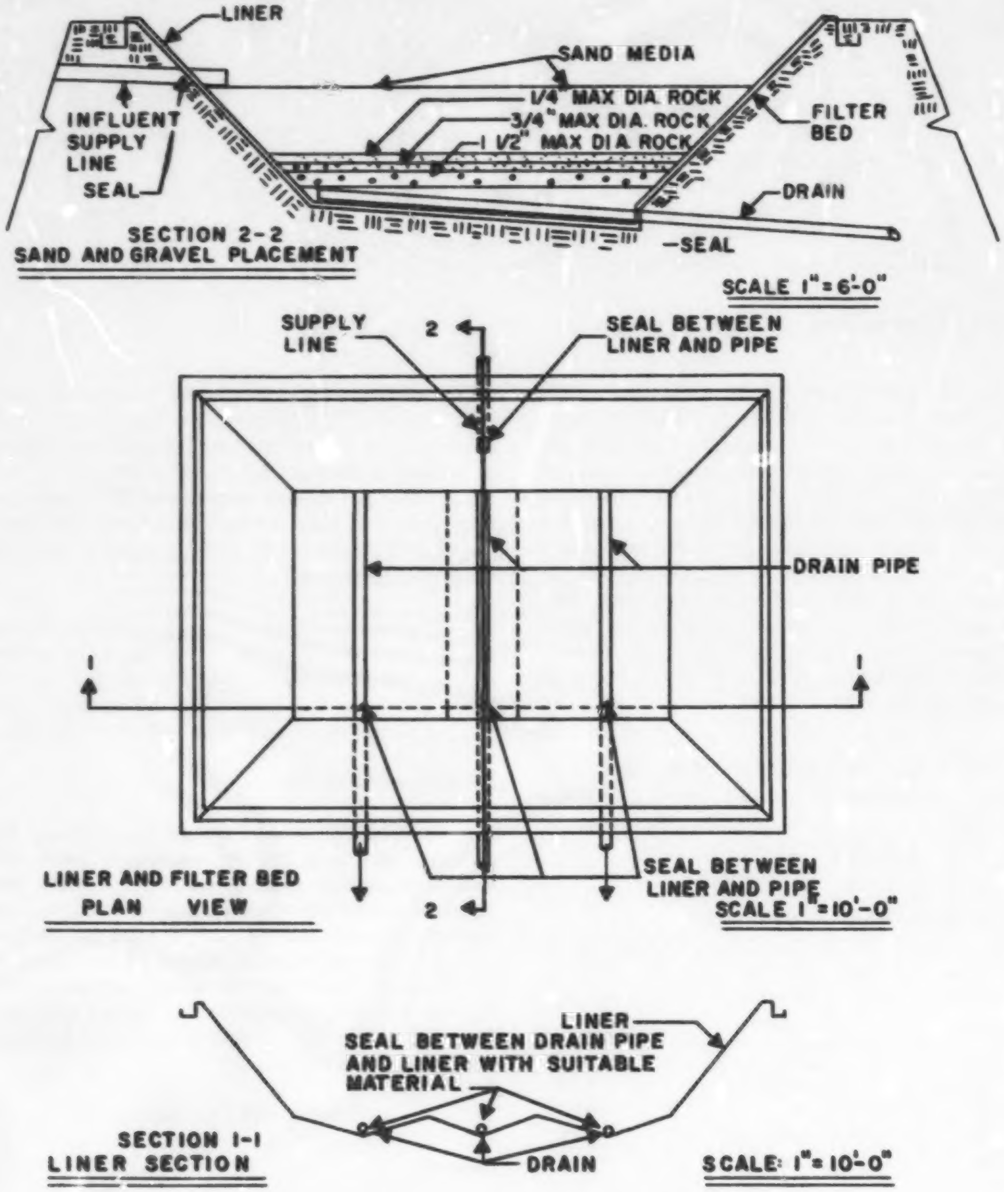
LINER
SAND MEDIA
1/4" MAX DIA. ROCK
3/4" MAX DIA. ROCK
1 1/2" MAX DIA ROCK
FILTER BED
INFLUENT SUPPLY LINE
SEAL
DRAIN
SEAL
SECTION 2-2
SAND AND GRAVEL PLACEMENT
SCALE 1"=6'-0"
SUPPLY LINE
2
SEAL BETWEEN LINER AND PIPE
DRAIN PIPE
1
1
LINER AND FILTER BED
PLAN VIEW
2
SEAL BETWEEN LINER AND PIPE
SCALE 1"=10'-0"
LINER
SEAL BETWEEN DRAIN PIPE AND LINER WITH SUITABLE MATERIAL
SECTION 1-1
LINER SECTION
DRAIN
SCALE: 1"=10'-0"

Figure 2. Cross section of a typical intermittent sand filter.

Table 2. Procedures for analysis performed.

Analysis	Procedure	Ref. No.
Biochemical Oxygen Demand	Standard Methods	APHA et al., 1971
Chemical Oxygen Demand	Standard Methods	APHA et al., 1971
Suspended Solids	Standard Methods	APHA et al., 1971
Volatile Suspended Solids	Standard Methods	APHA et al., 1971
Total Phosphorus	EPA Methods	EPA, 1971
Orthophosphorus	Strickland & Parsons (Murphy-Riley Technique)	Strickland & Parsons, 1968
Ammonia	Solorzano (Indophenol)	Solorzano, 1969
Nitrite	Strickland and Parsons (Diasotization Method)	Strickland & Parsons, 1968
Nitrate	Strickland & Parsons (Cadmium-Reduction Method)	Strickland & Parsons, 1968
Dissolved Oxygen	Standard Methods	APHA et al., 1971
Temperature	Standard Methods	APHA et al., 1971
pH	Standard Methods	APHA et al., 1971

Table 3. Average of all samples collected during the experimental period.

Loading Rate in mgad	BOD_5 mg/l	COD mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	Total Phosphorus mg/l	Ortho-Phosphate mg/l	NH_3-N mg/l	NO_2^--N mg/l	NO_3-N mg/l	pH	Temp. °C	Dissolved Oxygen mg/l	Algal Mass Removed Kg
Influent	8.1	69.7	26.1	16.9	2.082	1.754	2.469	0.025	0.100	8.8	23.2	4.2	[a]
0.2	2.4	42.1	6.8	0.9	1.756	1.695	0.166	0.083	4.670	8.0	23.6	6.2	21.988
0.4	1.7	27.8	3.7	1.0	1.595	1.458	0.197	0.025	4.936	8.1	24.4	6.1	21.538
0.6	2.3	27.5	5.5	1.4	1.767	1.644	0.293	0.055	4.985	8.0	23.7	5.9	26.108
0.8	3.5	40.6	7.2	0.7	1.863	1.683	0.322	0.090	4.372	7.9	25.0	5.7	18.783
1.0	2.8	39.8	7.1	1.0	1.980	1.717	0.486	0.160	3.664	8.1	25.1	5.0	19.008
1.2	4.3	49.7	4.8	0.8	1.776	1.657	0.541	0.154	3.383	8.1	24.0	4.8	12.336

[a] Not applicable.

identified as *Aphanocapsa sp.* The concentration of this particular alga has increased rapidly in recent weeks. The unknown unicellular green alga reported in Table 4 has been tentatively identified as the palmelloid state of *Chlamydomonas sp.*

The algal population is much different than that employed in previous laboratory and field scale intermittent sand filter experiments (Marshall and Middlebrooks, 1974). The algal population used in earlier experiments was predominantly *Chlamydomonas sp.*

Length of filter run

The length of filter run for each hydraulic loading rate is shown in Figure 3. As expected the length of filter run is directly related to the hydraulic

Table 4. Algae present in influent.

Alga	Cells per ml		
	Aug. 6	Aug. 9	Aug. 14
Blue Green Coccoid	33,920	92,928	100,188
Oscillatoria sp.	53	132	858
Microcystis sp.	0	0	330
Navicula sp.	8,480	5,016	17,688
Unknown Pennate	1,272	1,696	2,508
Pediastrum sp.	1,696	0	0
Schoideria sp.	53	66	396
Unknown Unicellular Green	0	0	1,254
Unknown Euglenoid	53	66	330
Total	45,527	99,904	123,552
Zooplankton	178	123	186

loading rate. The length of filter run varied from 14 days with a hydraulic loading rate of 1.2 mgad to 42 days with a hydraulic loading rate of 0.2 mgad.

These lengths of filter run are somewhat shorter than those reported earlier (Figure 4) for laboratory and pilot scale filters (Marshall and Middlebrooks, 1974). However, the average influent suspended solids concentration for previous investigations was 20.0 mg/l while for the present study the average influent suspended solids concentration was 26.1 mg/l with a range of 10.9 mg/l to 72.1 mg/l.

The time of day at which the filters are loaded may significantly effect the length of filter run. Filters which are loaded early in the morning and have influent standing on them throughout the entire daylight period may experience a tremendous amount of algal growth in the liquid above the filter itself. An experiment was conducted in which 6-inch diameter plexiglass columns were filled with filter influent and placed on the filter surface. The bottoms of the columns were sealed to prevent concentration of the algae as the water percolated through the sand and the water level in the column was held at the same level as the water on the filter by removing water from the column every hour. The experiment was conducted with three columns which permitted light penetration and three dark columns which did not permit light penetration to use as a control. The suspended solids concentration and the volatile suspended solids concentration of the columns were monitored with time. The results are recorded in Table 5 and shown graphically in Figure 5. The water remained on the surface of the filter for over 12 hours after loading and at the end of the daylight period approximately 1 foot of influent water remained on the filter surface. The suspended solids concentration had increased from 77.1 mg/l at 1 hour after loading to 222.4 mg/l at 12 hours after loading. This indicates that the average algal concentration filtered may have been 45 percent greater than the influent measurements indicate. Thus, the filter performance data presented in the report may be extremely conservative.

As shown by Figure 5 and Table 5, the increase in the volatile suspended solids concentration in the

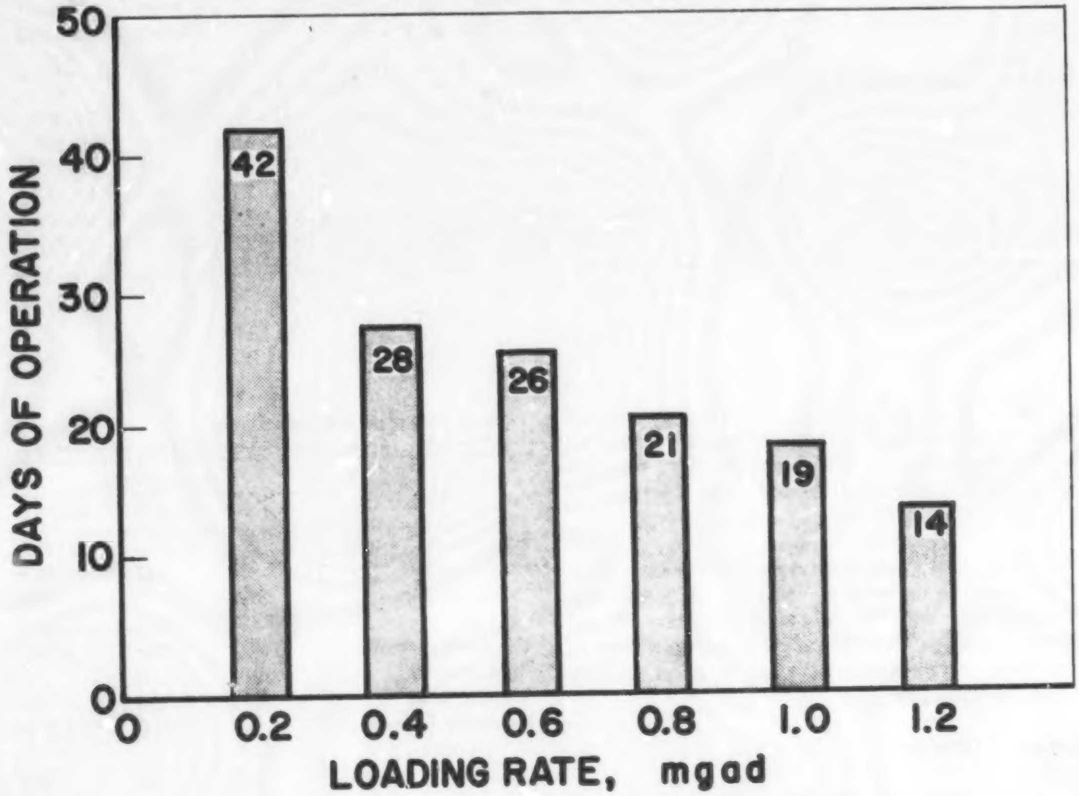

Figure 3. Days of operation before plugging.

liquid standing on the filter during daylight hours is similar to the increase in suspended solids concentration. The volatile suspended solids concentration increased from 55.29 mg/l at 1 hour after loading the filter to 109.03 mg/l at 12 hours after loading the filter. This represents an increase in volatile suspended solids concentration of 97 percent. This further indicates the conservative nature of the performance data presented in this report.

These results indicate that the length of filter run could be substantially increased by either loading the filters at night after sundown or by covering the filters to prevent photosynthesis.

An attempt was made to determine the actual mass of suspended matter removal by each filter before plugging. This was accomplished by multiplying the influent suspended solids concentration by the volume of liquid filtered by each filter daily. Because influent suspended solids were not monitored daily, it was necessary to extrapolate values for those days without influent suspended solids data by averaging the data of the day before and following the day without data. The results are reported in Table 3 and shown graphically in Figure 6. Admittedly, this is only an approximation, but it is superior to not analyzing the data.

Figure 6 clearly indicates that the 0.6 mgad loading rate removed a substantially greater total mass than did any of the other filters. A possible explanation for this phenomenon is that these filters are a biological system, and much of their treatment capability is related to aerobic bacteria (see section on nutrient removal). Thus, like any other biological system, there exists an optimum organic (nutrient) loading rate. Below this optimum rate, the organisms receive insufficient nutrients to establish a maximum population. Above this optimum rate, the organisms receive too much organic matter and anaerobic conditions exist which restrict filter performance.

Effluent quality with time

When the filters were initially placed in operation, the effluent suspended solids concentration (i.e., 34.6 mg/l at 0.4 mgad) far exceeded the influent

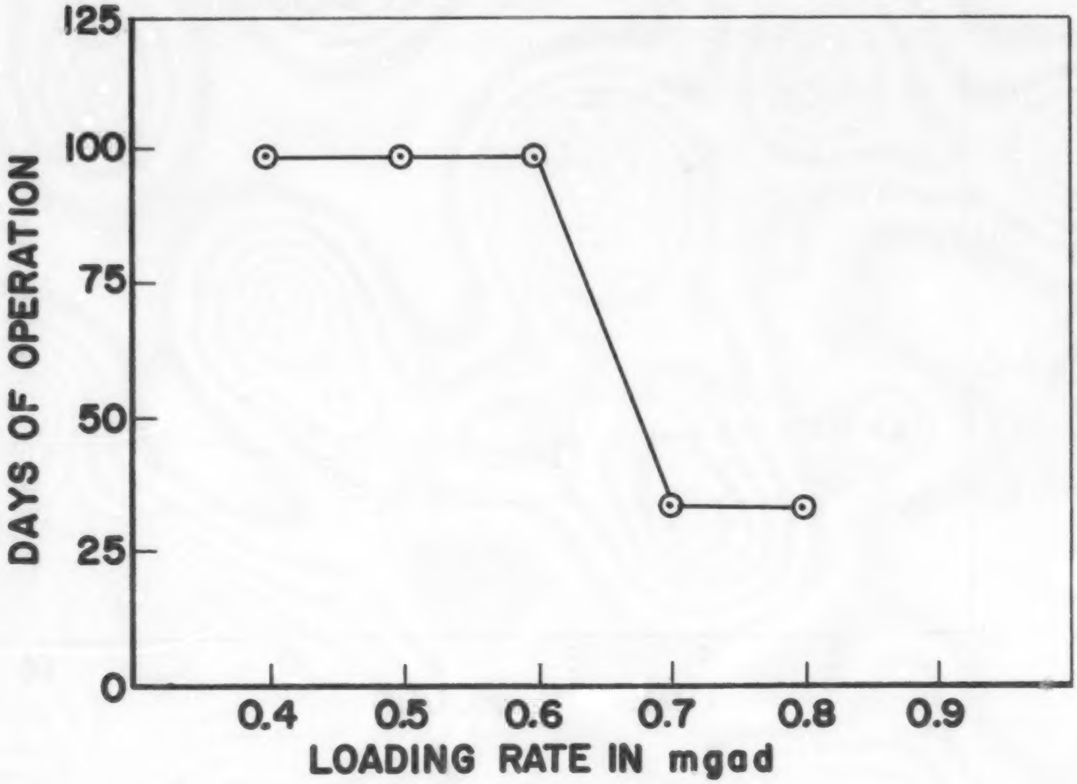

Figure 4. Days of operation for field scale filters with 0.17 mm sand.

Table 5. Algal growth on filters with time.

Time in Hours	Algal Growth[a]		Control[a]	
	Suspended Solids (mg/l)	Volatile Suspended Solids (mg/l)	Suspended Solids (mg/l)	Volatile Suspended Solids (mg/l)
1.0	77.1	55.29	75.2	49.99
2.3	81.3	56.32	81.4	55.44
3.6	92.9	62.33	77.4	50.55
5.0	90.0	63.19	73.6	49.23
6.0	93.3	66.98	73.1	53.24
8.0	102.0	74.94	69.2	49.23
10.0	164.0	80.48	68.9	48.12
12.0	222.4	109.03	78.9	52.19
Average	111.6	71.07	74.7	50.99

[a] Average of three columns.

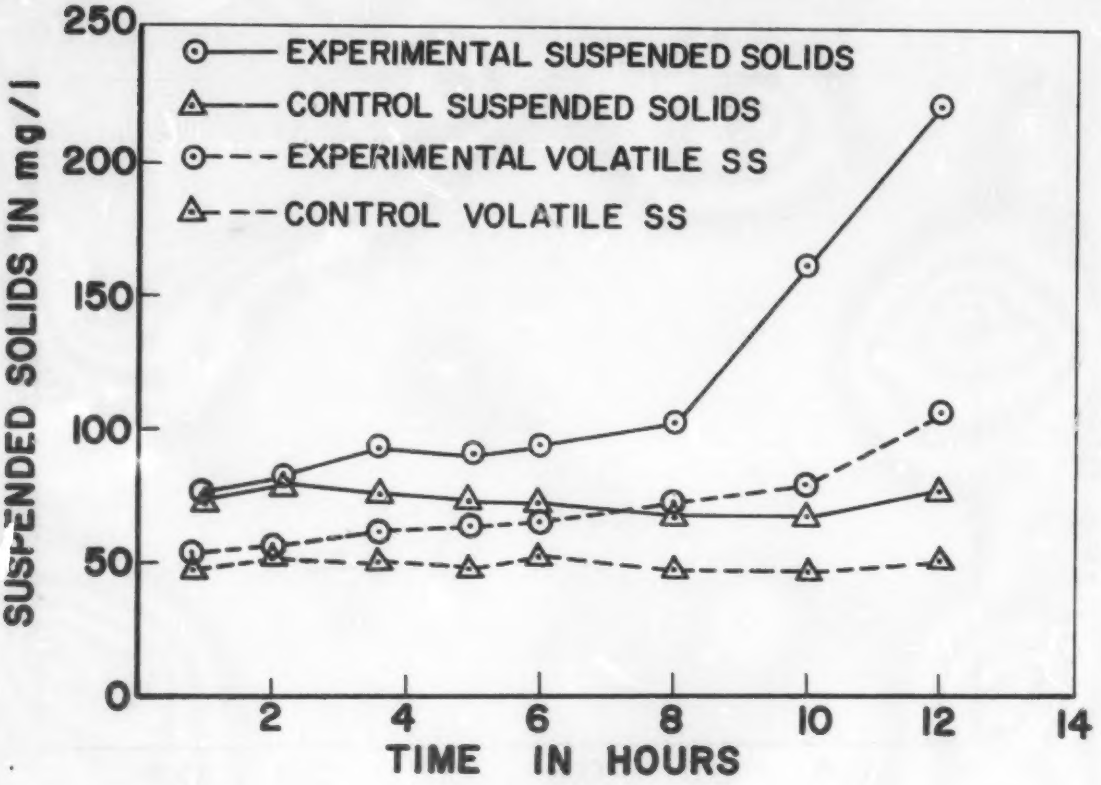

Figure 5. Algal growth on filters with time.

suspended solids (i.e., 24.9 mg/l). However, the effluent volatile suspended solids concentration (i.e., 2.5 mg/l at 0.4 mgad) was substantially less than the influent volatile suspended solids concentration (i.e., 17.8 mg/l). Essentially, the high initial effluent suspended solids concentration was composed of inorganic material being "washed" from the filters. This material had a fine sand texture and was probably very fine sand, dirt, and grit introduced into the filter system by the filter construction process. After several days of operation, this phenomenon ceased.

The effluent suspended solids of each filter were monitored with time after loading on three separate occasions to determine if variations existed. A typical plot is shown in Figure 7. It was found that the effluent suspended solids concentration peaks between 20 and 30 minutes after loading. This is probably due to the high velocity through the filter caused by the maximum head on the filter immediately following loading. However, on all effluent samples taken with time the volatile suspended solids concentration was less than 1.0 mg/l (influent VSS ≈ 17.0 mg/l). Thus, the variation in effluent suspended solids is probably caused by "filter washing." Because of this variation in effluent quality with time, it was decided to sample the effluent 2 hours after loading.

Suspended solids and volatile suspended solids

The average effluent suspended solids concentration for each hydraulic loading rate is reported in Table 3, and shown graphically in Figure 8. The average influent suspended solids concentration was 26.1 mg/l while the average effluent suspended solids concentration varied from 3.7 mg/l (0.4 mgad) to 7.2 mg/l (0.8 mgad). However, a maximum influent suspended solids concentration of 72.1 mg/l resulted in effluent suspended solids concentration of less than 4.0 mg/l for the 0.2 mgad and the 0.4 mgad hydraulic loading rates (all other filters were plugged at this time).

In general, the filtered effluent suspended solids concentration increased slightly with an increase in the hydraulic loading rate.

The average effluent volatile suspended solids concentration for each hydraulic loading rate is

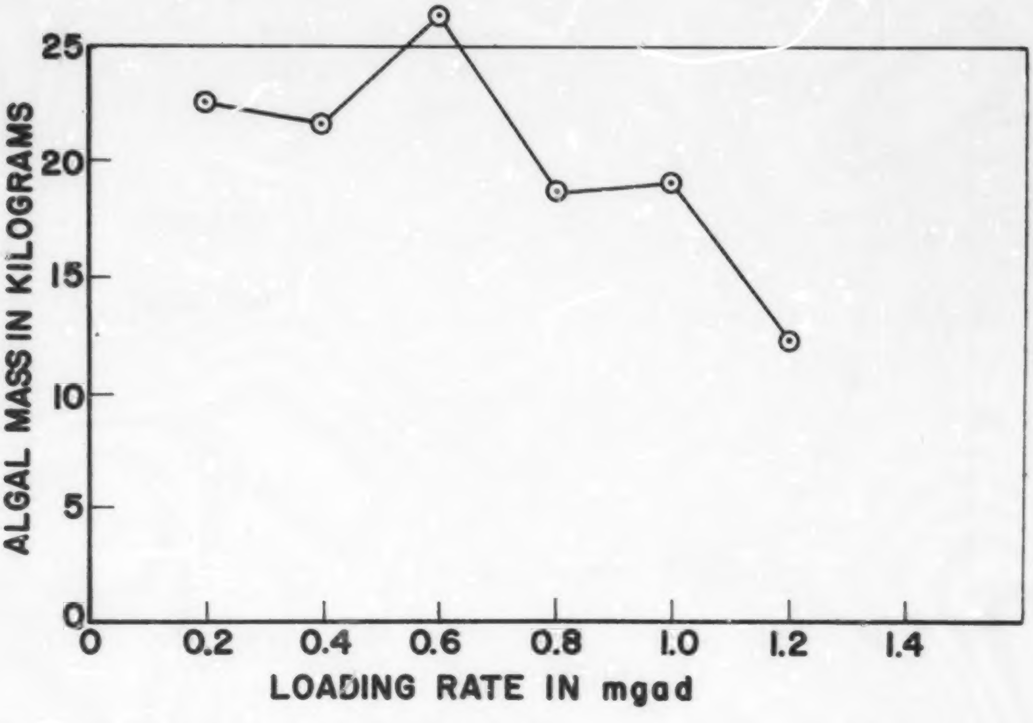

Figure 6. Total mass removed by each filter before plugging.

reported in Table 3, and shown graphically in Figure 9. The average influent volatile suspended solids concentration was 16.9 mg/l while effluent volatile suspended solids concentrations were all less than 1.5 mg/l. A maximum influent volatile suspended solids concentration of 63.4 mg/l resulted in an effluent volatile suspended solids concentration of 1.2 mg/l for both the 0.2 mgad and the 0.4 mgad loaded filters (all other filters were plugged at the time).

In general, the filtered effluent volatile suspended solids concentration appears to be independent of the hydraulic loading rate. Also, these results indicate that the filters remove over 90 percent of the applied volatile suspended solids.

BOD_5 and COD

The average biochemical oxygen demand (BOD_5) of the filtered effluent for each hydraulic loading rate is reported in Table 3, and shown graphically in Figure 10. The average influent BOD_5 was 8.1 mg/l while the average effluent BOD_5 ranged from 1.7 mg/l for a hydraulic loading rate of 0.4 mgad to 4.3 mg/l for a hydraulic loading rate of 1.2 mgad. The maximum influent BOD_5 applied to the filters was 14.3 mg/l with the corresponding effluent BOD_5 ranging from 1.3 to 2.8 mg/l.

In general, the filtered effluent BOD_5 increased with an increase in the hydraulic loading rate. However, such increases are relatively slight and may not be statistically significant.

The chemical oxygen demand (COD) of the filtered effluent for each hydraulic loading rate is reported in Table 3, and shown graphically in Figure 11. The average influent COD was 69.7 mg/l, while the effluent COD values changed from 27.5 mg/l (0.6 mgad) to 49.7 mg/l (1.2 mgad).

In general, the pattern of COD removal was similar to that for BOD_5 removal. The effluent COD appeared to increase slightly with increases in the hydraulic loading rate.

Ammonia, nitrite, nitrate

The ammonia, nitrite, and nitrate concentrations in the filtered effluent for each hydraulic

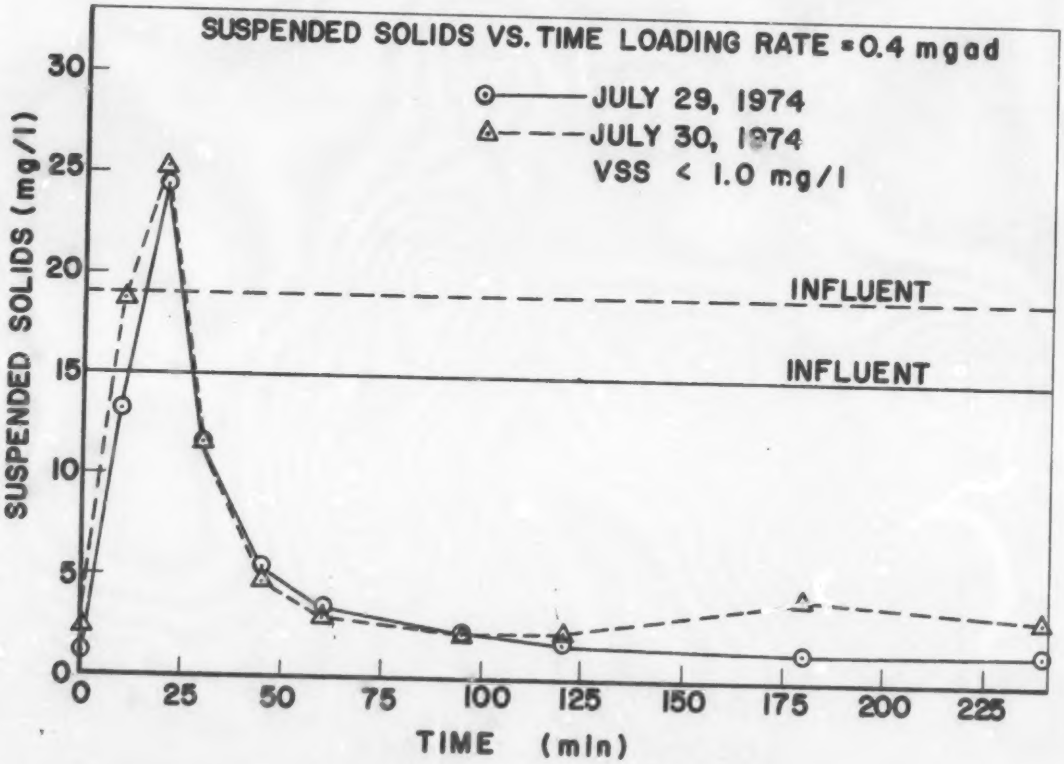

Figure 7. Filtered effluent suspended solids concentration with time.

loading rate is reported in Table 3 and shown graphically in Figures 12, 13, and 14. The influent ammonia-nitrogen concentration averaged 2.469 mg/l NH_3-N while effluent ammonia-nitrogen concentrations were all less than 0.541 mg/l NH_3-N. Figure 12 indicates that the effluent ammonia-nitrogen concentration increases with an increase in hydraulic loading rate. The influent ammonia-nitrogen is probably converted to nitrate-nitrogen by nitrification (see Figure 14) rather than being physically removed. Thus, the increase in effluent ammonia-nitrogen with hydraulic loading rate indicates that less nitrification occurs at the higher loading rates.

The decrease in the nitrification process at the higher hydraulic loading rate is evident in the nitrite-nitrogen and nitrate-nitrogen concentration of the filtered effluents. Figure 13 indicates that effluent nitrite-nitrogen almost doubled between the 0.2 mgad and the 1.2 mgad hydraulic loading rates. Figure 14 indicates a definite decline in effluent nitrate-nitrogen concentration with increased hydraulic loading rate.

The apparent decrease in nitrification with increasing hydraulic loading rates suggests that the bacterial population in the higher loaded filters is less efficient. This could be a result of organic overloading which may cause anaerobic conditions to exist within a portion of the sand filter bed and restrict the nitrification process.

Phosphorus

The values for both influent and effluent total phosphorus and orthophosphorus are reported in Table 3. Although effluent phosphorus concentrations are less than influent values, previous experiments (Marshall and Middlebrooks, 1974) have indicated that this is due to ion exchange within the filter sand bed and that once the ion exchange sites become saturated, phosphorus removal does not

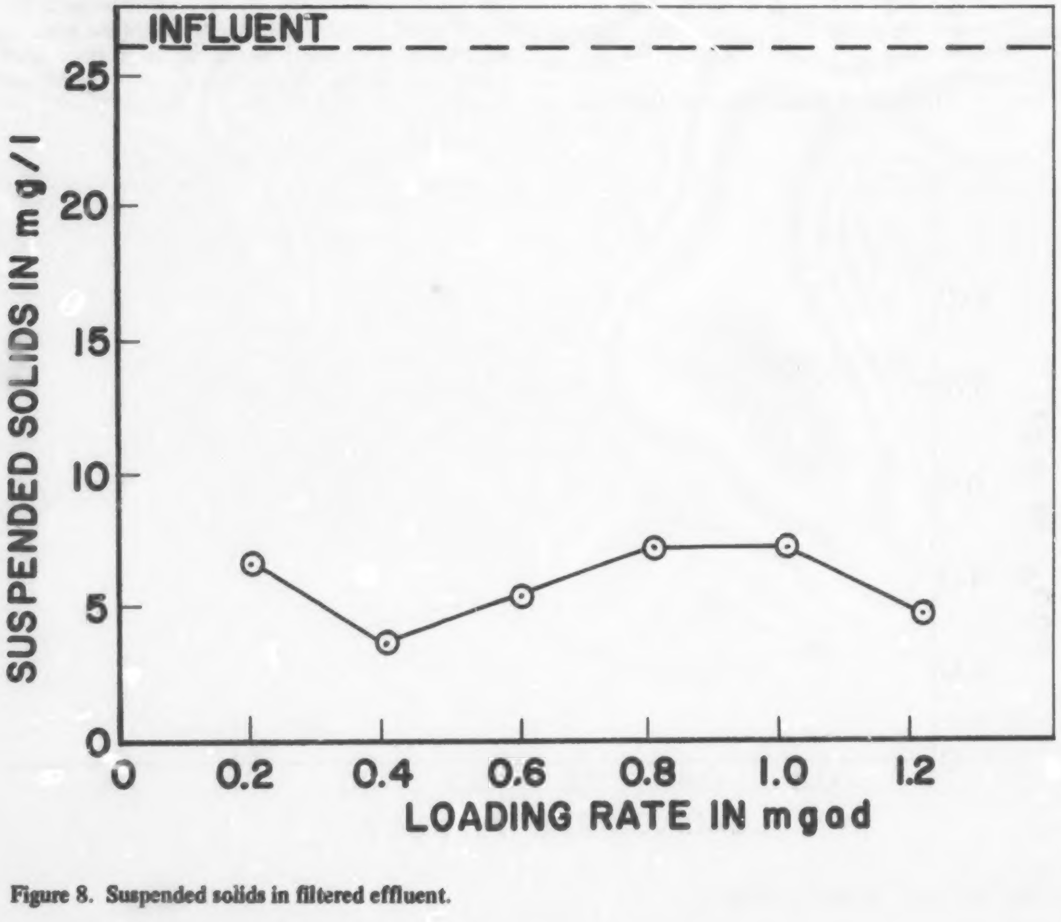

Figure 8. Suspended solids in filtered effluent.

occur. Thus, further operational experience is required before any definite conclusion on filter phosphorus removal performance can be justified.

Filter performance evaluation

A crude attempt to select the optimum hydraulic loading rate based on the information presented in this report is presented in Table 6. Seven parameters were selected to indicate overall filter performance. Each hydraulic loading rate was rated with a number between one and six. A rating of one indicates superior performance in a particular parameter while a rating of six indicates poor performance. In areas where two filters had equal performance the rank order was averaged.

The average rating for each filter was then determined without giving particular weight to any one parameter. The results indicated that the filters with a hydraulic loading rate of 0.4 mgad or 0.6 mgad were superior to the other hydraulic loading rates. However, such a selection must be viewed with extreme caution because of the limited data available for analysis and the selection process employed. The result could be substantially different if the selection parameters were weighted according to importance based on design, construction, and operational criteria.

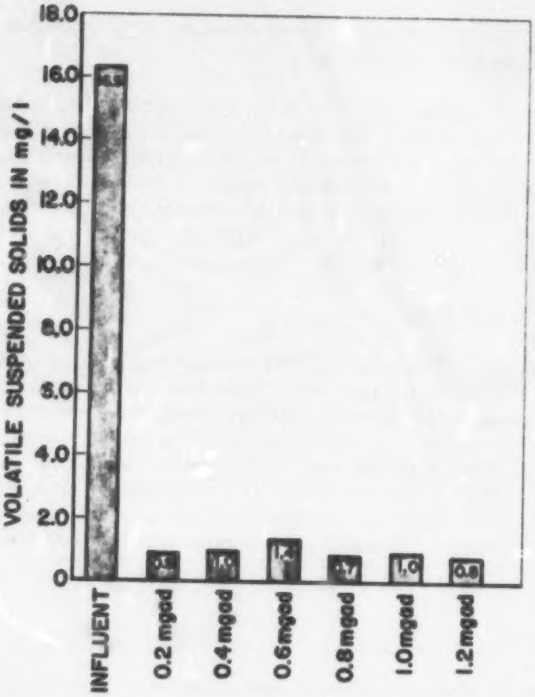

Figure 9. Filter volatile suspended solids performance.

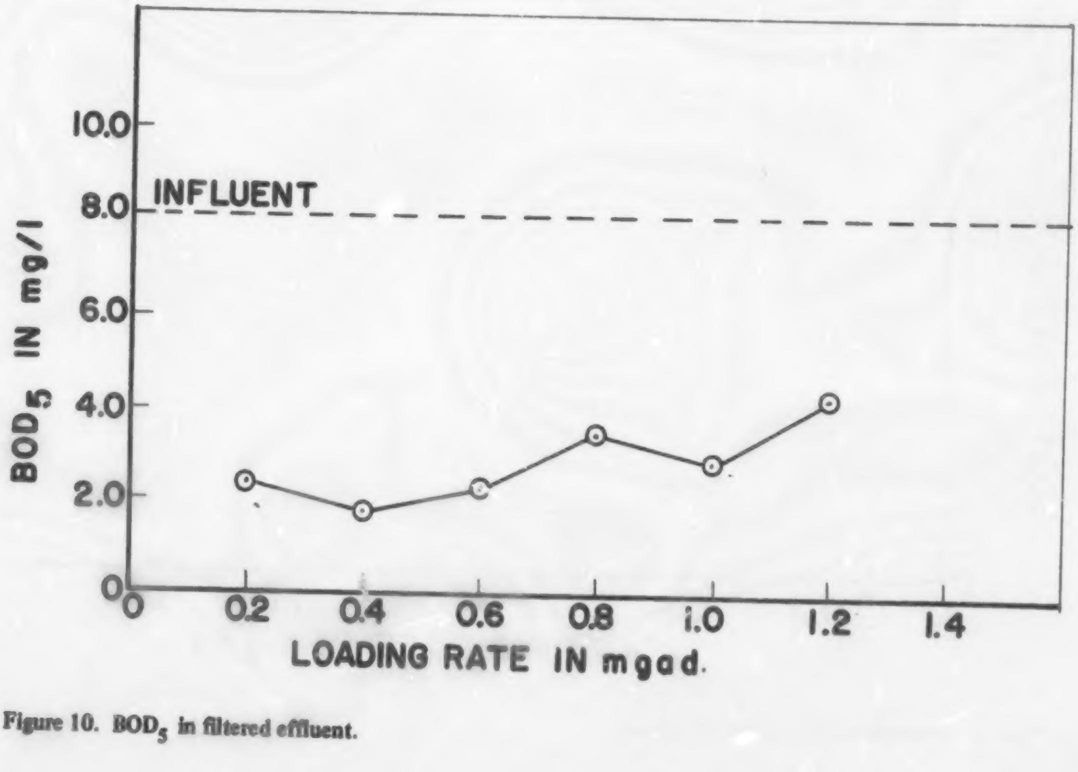

Figure 10. BOD_5 in filtered effluent.

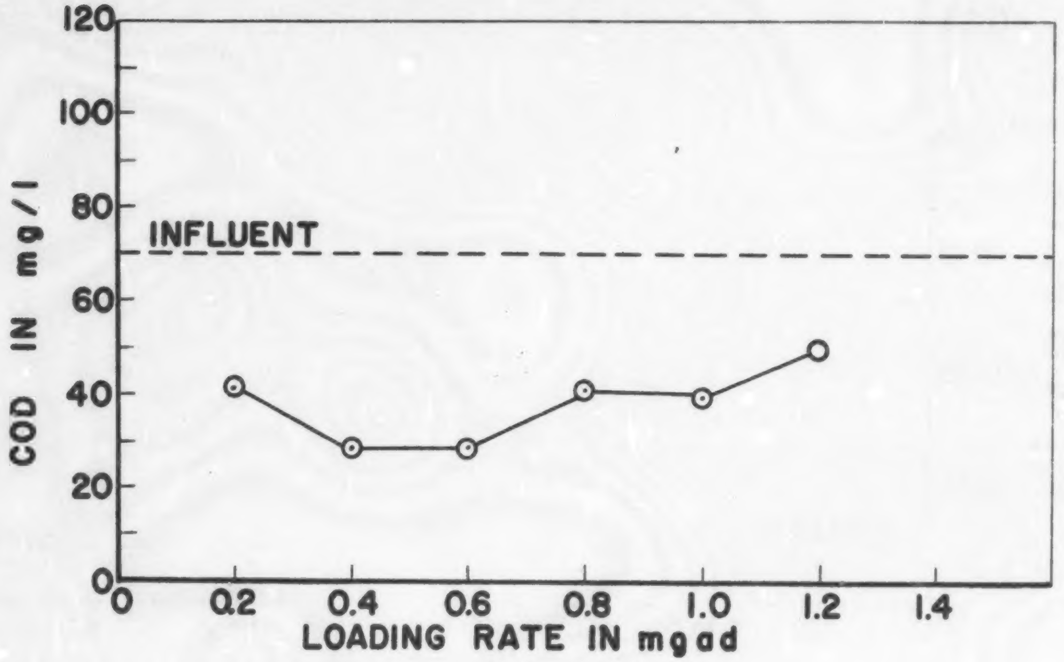

Figure 11. COD in filtered effluent.

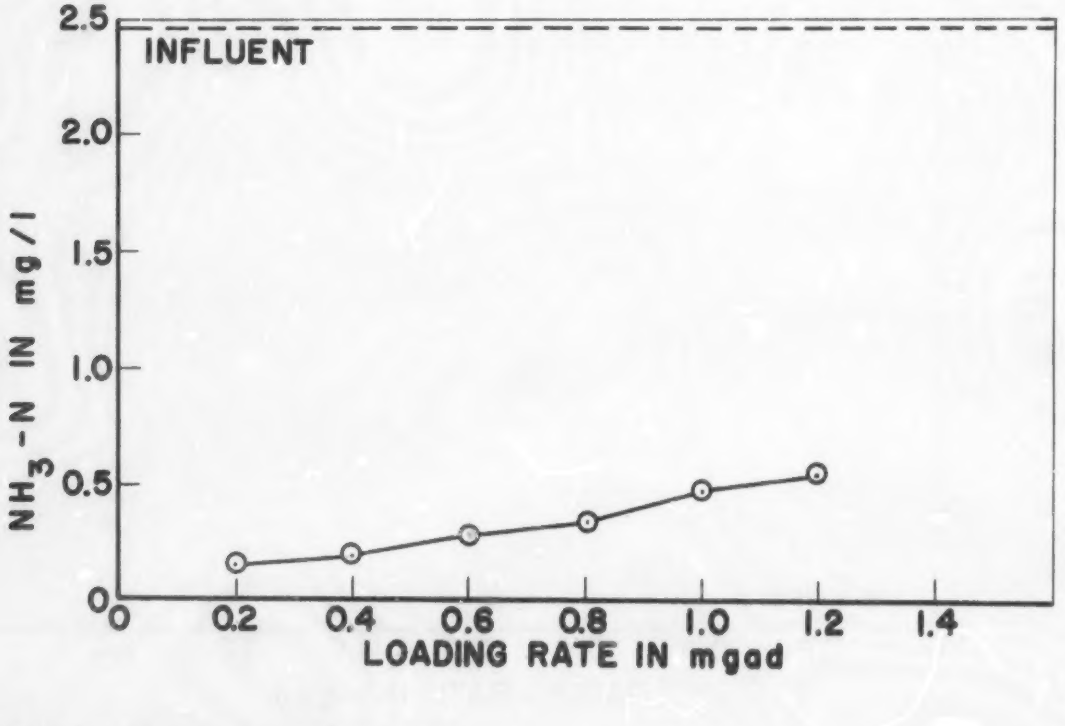

Figure 12. Ammonia nitrogen in filtered effluent.

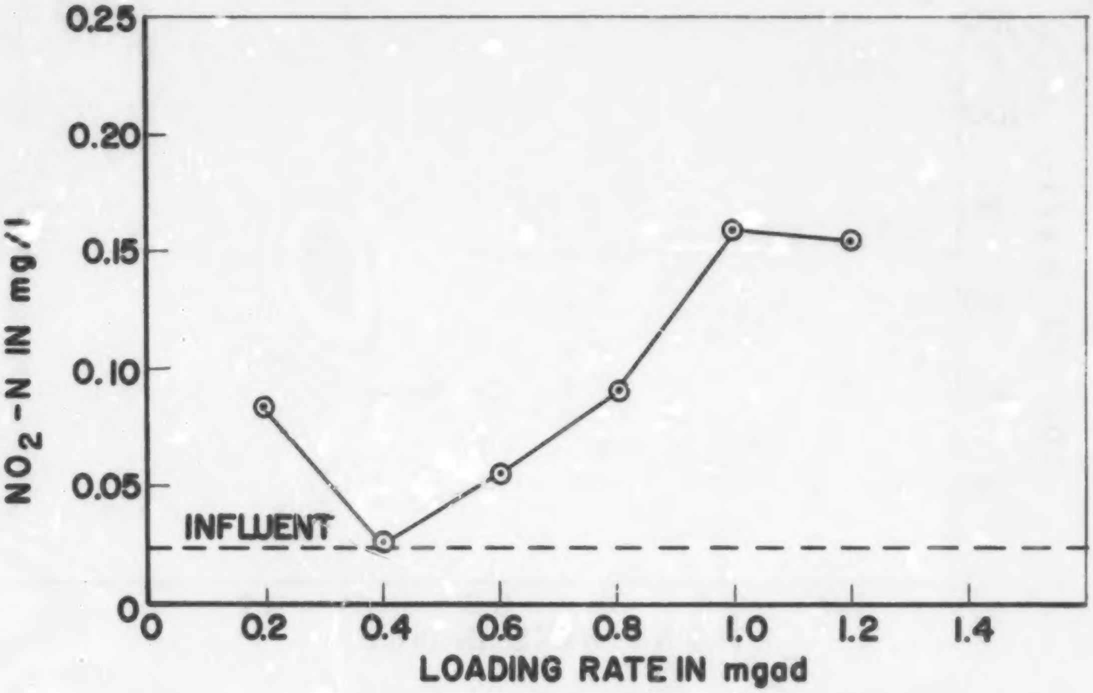

Figure 13. Nitrite nitrogen in filtered effluent.

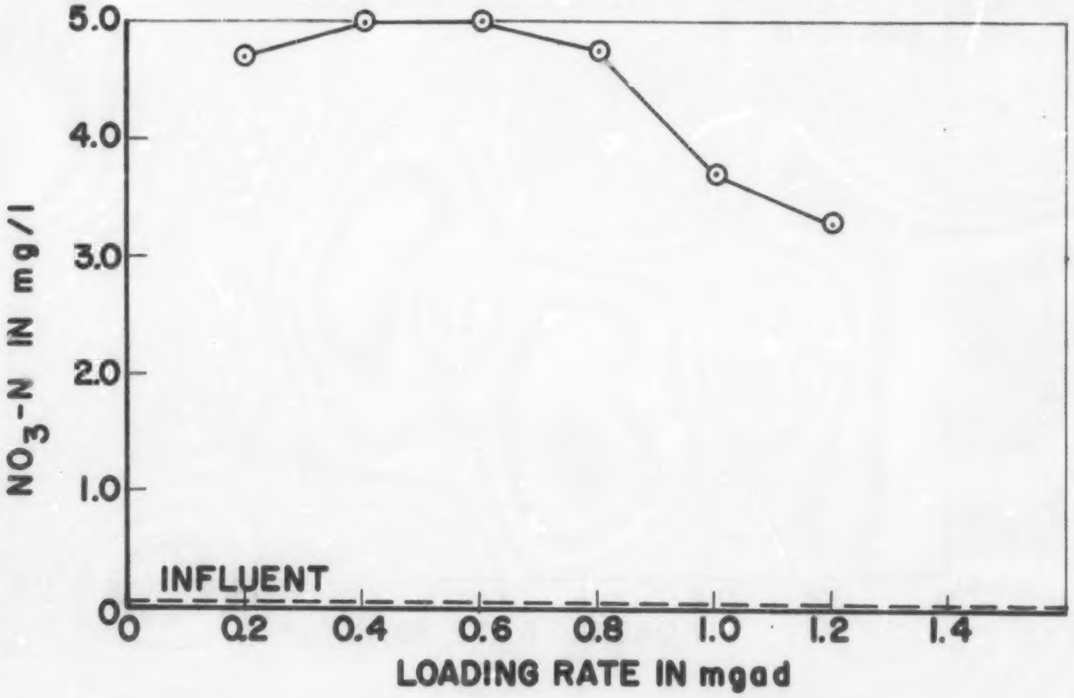

Figure 14. Nitrate nitrogen in filtered effluent.

Table 6. Overall filter performance evaluation.

Loading Rate in mgad	BOD_5	COD	SS	VSS	NO_3-N	Algal Mass Removal	Hydraulic Loading Rate	Average Ranking
0.2	3	5	4	3	3	2	6	3.71
0.4	1	2	1	4.5	2	3	5	2.64
0.6	2	1	3	6	1	1	4	2.57
0.8	5	4	6	1	4	4	3	3.85
1.0	4	3	5	4.5	5	5	2	4.07
1.2	6	6	2	2	6	6	1	4.14

Comparison With Previous Results

The full-scale filters have basically duplicated the performance of the laboratory and pilot scale units (Marshall and Middlebrooks, 1974). In all cases when the BOD, SS, VSS, and nutrient concentrations applied to the systems were approximately equal, removals obtained at various hydraulic loading rates are practically indistinguishable between the laboratory, pilot, and full-scale filters.

Fines were washed from the full-scale filter media for a considerable period of time as was the case with both the laboratory and pilot scale units. This accounted for the majority of the solids in the effluents from the filter units. Volatile suspended solids averaged less than 1 mg/l in the effluent from the full-scale filters at all six loading rates employed; whereas, in the laboratory and pilot units the effluent VSS concentrations frequently exceeded 4 mg/l. Whether this difference was caused by the characteristics of the genera of algae being removed or is attributable to variations in laboratory technique cannot be determined. However, there were significant differences in the genera of algae present.

During both the laboratory and pilot studies, *Chlamydomonas sp.* were the predominant genera in the lagoon effluent with only an occasional diatom (5 percent) or *Anabaena sp.* (5-20 percent). At no time were there less than 75 percent *Chlamydomonas sp.* in the effluents applied to the filters. The full-scale filters received a lagoon effluent containing mostly a blue-green coccoid alga, tentatively identified as *Aphanocapsa sp.* and *Chlamydomonas sp.* were present in only small amounts (see Table 4). Because the performance of the full-scale filters was in most cases superior to the laboratory and pilot units, it appears that algal genera have little impact on the intermittent sand filter. The *Aphanocapsa sp.* are much smaller than the *Chlamydomonas sp.* and do not agglomerate; therefore, a large percentage of the *Aphanocapsa sp.* would be expected to penetrate and pass the filters. However, this did not occur with the .170 mm effective size sands used in the full-scale filters.

BOD_5 removals were essentially the same for all three size systems, and an effluent containing less than 5 mg/l was obtained at all hydraulic loading rates. The mean influent BOD_5 concentrations applied to the laboratory and pilot filters were approximately 50 percent greater (13 mg/l versus 8 mg/l) than that applied to the large units, but approximately equal quality effluents were produced. Apparently, as in other biological systems, there are certain compounds which are difficult to biodegrade during the short residence time in the filters. Consequently, a minimum effluent BOD_5 concentration of approximately 2 mg/l can be obtained with the intermittent sand filter.

The pH values of all filter effluents were approximately 8.0, but ranged from 7.9 to 8.9. The value was influenced greatly by the characteristics of the lagoon effluents.

Effluent D.O. measurements were not performed on the laboratory and pilot units. In the full-scale unit the D.O. concentration of the effluents were directly related to the loading rates. Up to a loading rate of 0.8 mgad the effluent D.O. concentration was greater than 5.5 mg/l as required in the Utah Class "C" standard. Above 0.8 mgad the effluent D.O. concentration dropped below 5.0 mg/l in several samples.

Because of the differences in applied algal concentration, a direct comparison of the length of run obtained with the various size units is difficult. However, at common hydraulic loading rates and approximately equal algal concentrations, the units operated about equal periods of time before plugging. Approximately one month of continuous operation at the peak algae production in the lagoons can be expected with a filter containing sand with an effective size of 0.17 mm. The larger surface areas exposed to the sunlight in the large filters introduced a variable not encountered in the laboratory and encountered to a more limited extent in the shaded pilot units. A significant increase in algal concentrations due to growth after the lagoon effluent was applied to the filters probably accounted for the slightly shorter filter runs obtained in the full-scale units. The same asymptotic relationship developed in all size units when the days of operation were plotted versus the hydraulic loading rate. A loading rate of approximately 0.5 mgad continues to appear to be the optimum based upon solids and BOD_5 removals and period of operation between cleanings.

The large units have so far confirmed the conclusions based upon the laboratory and pilot scale system. Intermittent sand filtration treating secondary lagoon effluent is capable of consistently producing an effluent with VSS and BOD_5 concentrations of less than 5 mg/l, and provides operational simplicity and economy.

Series Intermittent Sand Filter Performance

A study is currently underway at the Utah Water Research Laboratory, Utah State University, to determine the feasibility of operating intermittent sand filters with different effective size sands in series. At present three pilot scale (16 sq ft surface area each) series filter operations are under study. Each filter operation consists of three filters in series with the initial filter having an effective size sand of 0.72 mm, the intermediate filter sand effective size is 0.4 mm, and the final polishing filter has an effective size sand of 0.17 mm (see Figure 15). Hydraulic loading rates for the three systems are 0.5 mgad, 1.0 mgad, and 1.5 mgad. Preliminary results indicate that

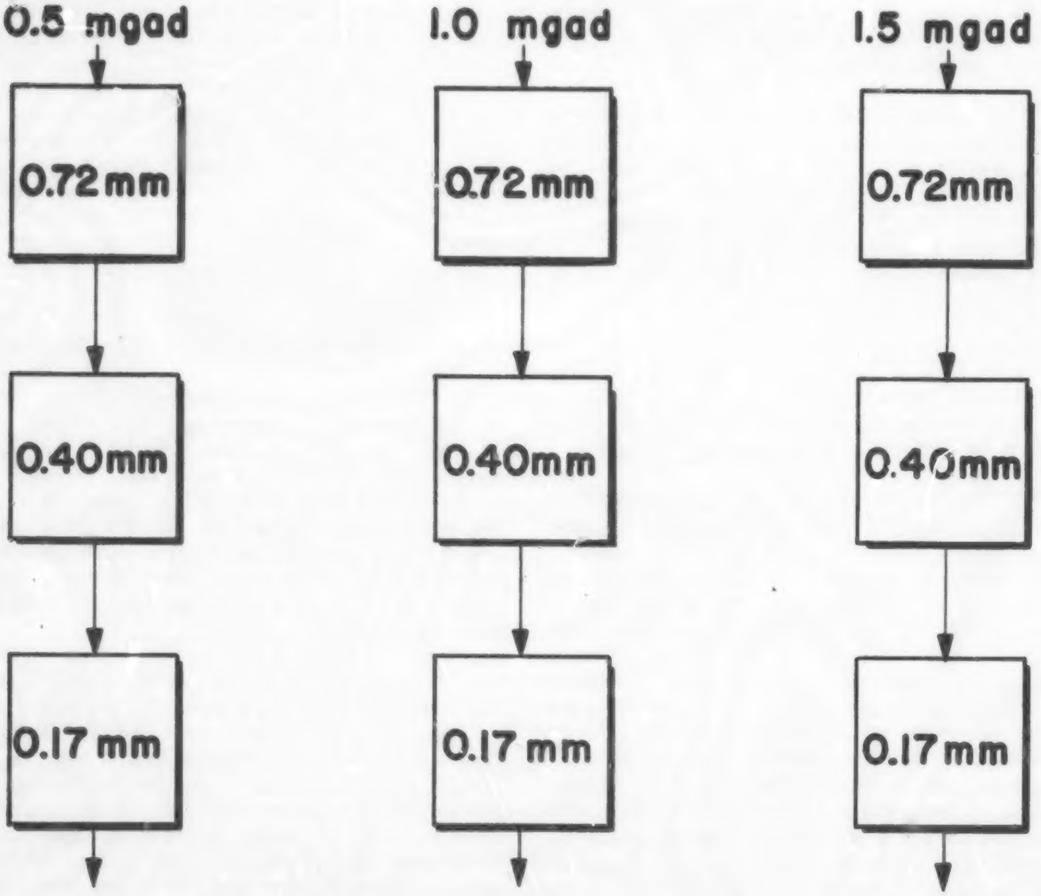

Figure 15. Series intermittent sand filtration operation.

a high quality effluent is produced and that the length of filter run is substantially increased.

Summary and Conclusions

Wastewater lagoons provide simple, economical, and low maintenance waste treatment for many small communities. However, the degree of treatment possible with lagoons may be inadequate to meet future discharge standards. Therefore, a definite need exists to develop a system which can simply and economically upgrade lagoon effluents to a level which satisfies future discharge requirements.

A full-scale experimental intermittent sand filtration system has been constructed at the Logan Municipal Sewage Lagoons by the Utah Water Research Laboratory, Utah State University. The objective of the project is to evaluate the performance of intermittent sand filters in upgrading the quality of wastewater lagoon effluent. Data have been collected and analyzed from July 12, 1974, to August 22, 1974, for hydraulic loading rates ranging from 0.2 mgad to 1.2 mgad.

Based on the information presented in this article and previous studies (Marshall and Middlebrooks, 1974) the following conclusions can be made.

1. The algal genera present in the lagoon effluent have little impact on performance of the intermittent sand filters.
2. Length of filter run is directly related to influent algal concentration and hydraulic loading rate.
3. The full-scale filters had a slightly shorter length of filter run than did previous laboratory and pilot scale filters.
4. The length of filter run observed in the full-scale filters was substantially affected by algal growth in the liquid above the filter.
5. The length of filter run varied from 14 days with a hydraulic loading rate of 1.2 mgad to 42 days with a hydraulic loading rate of 0.4 mgad.
6. The filter with a hydraulic loading rate of 0.6 mgad removed over twice as much total suspended matter as did the filter with a hydraulic loading rate of 1.2 mgad.
7. It is necessary to allow a period of time for the filters to "wash" out fines introduced during construction, before a low effluent suspended solids concentration can be achieved.
8. With an average influent suspended solids concentration of 26.1 mg/l, all filters produced an effluent suspended solids concentration of less than 7.2 mg/l.
9. Effluent suspended solids concentration increased slightly with an increase in the hydraulic loading rate.
10. With an average influent volatile suspended solids concentration of 16.9 mg/l, all filters produced an effluent with less than 1.5 mg/l of volatile suspended solids.
11. The filtered effluent volatile suspended solids concentration is independent of hydraulic loading rate.
12. The intermittent sand filter can produce a consistent effluent BOD$_5$ of less than 5.0 mg/l.
13. Filtered effluent BOD$_5$ increases slightly with an increase in the hydraulic loading rate.
14. Filtered effluent COD increases slightly with an increase in the hydraulic loading rate.
15. Influent ammonia-nitrogen concentrations were reduced from 2.469 mg/l NH_3-N to less than 0.541 mg/l NH_3-N by all hydraulic loading rates.
16. The full-scale intermittent sand filters essentially confirm the previous laboratory and pilot scale studies.
17. Experiments are currently being conducted to evaluate the performance of series intermittent sand filter operations.

Acknowledgments

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Dr. Ronald F. Lewis has been the Project Officer supervising this contract for the Environmental Protection Agency.

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PERFORMANCE OF RAW WASTE STABILIZATION LAGOONS IN MICHIGAN WITH LONG PERIOD STORAGE BEFORE DISCHARGE

D. M. Pierce[1]

Background

In recent years the prevalance and popularity of the waste stabilization lagoon have grown in many sections of the country. Many additional projects of this kind are now under construction or in the planning stage, largely in very small communities. Some, however, are proposed for flows of up to one million gallons per day. Although several thousand municipal projects of this kind are in operation today, and generally considered to be performing quite satisfactorily, very little factual information has been assembled on their performance. So today, with the adoption of regulations calling for a minimum of secondary treatment as specifically defined and future requirements for "best practicable treatment" there is an urgent need to assess the capability of this method of treatment when opeated under a variety of conditions.

Information collected at municipally-owned and operated lagoons in Michigan during the past 5 years indicates that systems with relatively low organic loadings, long storage periods, and twice-yearly discharge hold promise of meeting the requirements for secondary treatment. The special EPA work group on waste stabilization lagoons has expressed interest in exploring the efficacy of this method and how it can be managed most effectively to achieve maximum performance dependably. Accordingly a study was undertaken for this purpose.

The person conducting this study is intimately familiar with the Michigan program, having been responsible for state regulatory control of design and operation of municipal wastewater facilities for many years until last July. This relationship assured the full cooperation and support of both regulatory agency personnel and municipal officials and employees in assembling and interpreting the essential information. Many conferences were held with division engineers. Visits were made to operating facilities in company with operation personnel. Many people performing the analyses on the lagoon effluent samples provided valuable and timely information, particularly on samples recently analyzed.

Facilities Studied

To obtain maximum available pertinent information it was decided to assemble information on all municipal raw waste stabilization lagoons discharging effluents during the past 2 years. Presently there are about 125 municipal lagoon systems in the state. Many have not had their first discharge by reason of the recency of their completion, or because of soil conditions favoring high on-site percolation rates. A few of the facilities have some aeration facilities. Others were designed for percolation into the ground. Neither of these were included in the study.

It was found that 49 of the systems are representative of the selected pattern of long storage and twice-annual discharge. Each of these have sufficient operational data to permit a meaningful evaluation of performance.

Although there are quite a range in design features and operational methods, all conform to a general pattern adopted by the state regulatory agency several years ago as part of the overall pollution control plan related to water quality standards.

Essential Features of Michigan Lagoon Program

The following information is provided to acquaint the reader with pertinent background on concepts relating to loadings, construction features and management methods used by operating personnel and the relationship between the operator and the state regulatory agency. This should be helpful in the interpretation and understanding of the data collected and analyzed in this study.

Design features

Generally, the principles of design specified in Chapter 90 of the *Recommended Standards for Sewage Works* adopted by the Great Lakes-Upper

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[1] D. M. Pierce is special consultant for the U.S. Environmental Protection Agency.

Mississippi Board of State Sanitary Engineers (Ten States' Standards) have been incorporated in these projects. In all cases the design loading has been approximately 100 persons (or PE) and 20 lbs BOD_5 per acre. Inlet, outlet, and interconnecting piping usually permits control of raw sewage into any cell and the discharge of treated wastewaters from any cell. Valving permits a variety of flow patterns with capability to isolate any cell or combination of cells. Sealing of the bottom and side slopes is provided in all of these systems to permit control of water depths within practical limits. Clay is most commonly used although bentonite has been mixed with resident soils at a few locations.

Apportionment of surface area and capacity among the cells in the system varies considerably. Maximum operating water depth usually is about 6 feet plus or minus 1 foot. (See Table 1 for capacity and discharge data.) Quite commonly cells are equal in area and volume. Some have equal area in each cell, but different maximum operating water depths. Usually the shape, area, and depths depend to some extent on topography and shape of available property, relative elevation of inlet sewer and point of lagoon discharge, depth to rock or water table, cost of earth moving, and related considerations.

Great care is taken in the earth work during construction to provide good mixing of the sealing materials with native soils, a level bottom free of debris or organic material and a well aligned shoreline. Slopes are seeded or sodded to reduce erosion and to permit grass mowing and weed control.

Discharge control facilities are designed to permit selection of any desired elevation for discharge, ranging from highest water elevation (automatic overflow level) to within a foot or so of the bottom.

All of these systems were designed for semi-annual discharge, with retention of total flow from the sewer system from late November until about April 15 and from about May 15 to October 15. It is expected that the contents will be discharged during a period of 2 to 4 weeks depending on the quantity stored; rates of percolation, precipitation and evaporation; hydraulic capacity limitations of the piping system; and the quality of the effluent in relation to conditions in the receiving water body. This leaves some 150-170 days for summer and winter storage without discharge. Since in Michigan annual precipitation and evaporation are about equal, the volume accumulated in 160 days between discharge periods would increase water depth by about 5 feet, assuming an average flow of 100 gallons per capita per day and no percolation. If the rate of percolation is 1/8 inch per day, the 5-foot increase in water depth would be reduced by 20 inches leaving 3 to 3½ feet of depth to be discharged. Experience has shown that actual accumulations may range from 1 foot to as much as 8 feet in the theoretical 160–day period. In some instances combinations of low per capita flow rates, high percolation, high evaporation and low precipitation rates result in such a low increase in water volume that no discharge is needed or desirable in one of the normal semi-annual discharge periods. There are several situations of this kind, particularly in the early years of operation, where no discharge is made for up to two years. These are not included in this study. In other circumstances where per capita flow is high, usually by reason of high inflow-infiltration rates, minimal percolation and high precipitation-evaporation ratios, the rate of accretion in the lagoons is so high that discharge periods in either or both spring and fall must be extended over a period of 30 days or more after accumulation to the maximum safe water elevation.

Operation and maintenance practices

The state regulatory agency requires each municipality to delegate full responsibility for lagoon operation and maintenance to one person on its public works staff. Wastewater Division personnel work with this person and other members of the work force in a continuing consultation and surveillance program, including both assembled and on-the-job training and advice. A manual on operation and maintenance principles is furnished each local operating agency for general guidance in addition to the manual which EPA now requires for each funded project.

Regular operation routine involves daily visitation and maintenance of lift stations. Usually the lagoon site is visited each day of the 5-day working schedule to observe conditions and perform essential operations. All lagoon properties are fenced and have locked gates. On each visit the operator drives his pickup truck on the roadway atop the dykes making a complete tour of the lagoons to observe their condition. He looks for signs of burrowing rodents, surface scum, sludge buildup near inlets, emergent weeds, and evidence of erosion. Action is taken as soon as possible to correct any adverse conditions. Grass and weeds on dyke slopes and tops are mowed when needed to assure a dense growth for erosion control. Brush and weeds at the shoreline are removed or otherwise controlled to prevent spreading which would interfere with natural waste treatment processes. Attention is given to management of flow into and between cells. In some systems normal flow patterns consist of the raw waste entering Cell No. 1 and flowing to Cell No. 2 through the connecting pipe located near the bottom. Flow from Cell No. 2 to Cell No. 3 is similar to this. In other systems differences in elevation require manual regulation of flow between the cells by the operation of valves. From time to time conditions require re-routing of flow for water depth adjustment, temporary resting of a cell, repair work, etc. Stop gates, valves, and control structures are examined occasionally and maintained or repaired if needed.

Records and reports

Observations are recorded routinely on standard monthly operation reports furnished by the state regulatory agency, Forms D-4.9 and D-4.9a. Sewage flow quantities (where measured), power consumption, and weather data are recorded daily. Lagoon liquid depth, dissolved oxygen, and ice conditions are recorded at least one day per week during the storage season and daily during discharge. Flow patterns among the lagoons and effluent discharge are to be recorded regularly on Form D-4.9a.

Discharge control program

Discharge of effluents follows a consistent pattern for all lagoons. The following steps are usually taken:

1. Isolate the lagoon to be discharged, usually the final one in the series, by valving-off the inlet line from the preceding lagoon.

2. Arrange with a nearby community to analyze samples for chemical analysis including BOD, suspended solids, volatile suspended solids, pH, and frequently for total phosphorus.

3. Plan work so as to spend full time on control of the discharge throughout the period.

4. Sample contents of lagoon to be discharged for dissolved oxygen, noting turbidity, color and any unusual conditions.

5. Note conditions in the stream to receive the effluent.

6. Notify Basin Engineer of Wastewater Division of the state regulatory agency of results of these observations and plans for discharge and obtain his approval.

7. If discharge is approved, proceed as follows: Commence discharge and continue so long as weather is favorable, dissolved oxygen is near or above saturation values and turbidity is not excessive following the prearranged discharge flow pattern among the lagoons. Usually this consists of drawing down the last lagoon in the series, or last two if there are three or more, to about 18-24 inches after isolation, interrupting the discharge for a week or more to divert raw waste to the last lagoon (or to No. 2 if there are three or more) and resting the No. 1 lagoon before its discharge. When this lagoon is drawn down to about 24 inches or so the usual series flow pattern is resumed.

During discharge to the receiving waters samples are taken at least three times each day near the discharge pipe for immediate dissolved oxygen analysis. Composite samples of the lagoon effluent are collected, usually 5 days per week, at points designated by the Basin Engineer (see 6 above) for chemical analyses. Each such sample is composed of at least three equal volume portions collected morning, noon, and late afternoon. Samples are refrigerated during collection and delivered within 24 hours (usually same day) to the laboratory conducting the analyses. The analyst must have demonstrated capability to perform the analysis. All work is performed in accordance with *Standard Methods* on unfiltered samples.

At least one grab sample per day, 5 days per week, is collected for analysis of fecal and total coliforms in sterilized bottles furnished by the State Department of Public Health. The standard bacteriological analysis form F216a is used to record pertinent information and is mailed with the bottle to the department for analysis.

The person performing the chemical analyses furnishes the operator and the regulatory agency a copy of the test results when all analyses for the seasonal discharge have been completed, using Form R4613. Coli test results are recorded on the *Standard Bacti* form and furnished to the Wastewater Division and the operator.

The Study and What It Reveals

The prime objective of this study is to determine whether and under what conditions the secondary treatment requirements can be met by raw waste stabilization lagoons designed and operated in the manner outlined in the foregoing discussion. Attention was given to factors which might affect the level of performance, as reflected in effluent quality. Factors considered were effect of differences in population loadings per acre or unit volume; number of cells; length of discharge period; comparison of spring with fall discharges; effect of ice cover and low temperatures without ice; relationship of BOD_5 to suspended solids under varying conditions; and trends and levels of coliforms under varying conditions; and relationship of total coli to fecal coli.

Study methods

The cooperation and assistance of the Municipal Wastewater Division, Department of Natural Resources, was solicited in providing access to all pertinent records, reports, and design data. This was provided in full measure in many most helpful ways. Discussions with division staff members assisted greatly in identifying the lagoons which have discharged effluents during the past 2 years and in obtaining desired information on design features, population loadings, presence of industrial wastes, identification of operating personnel and any unusual problems known to exist which might affect performance. All monthly operation reports and discharge reports were made available for review. Many were discussed with the responsible basin engineer for clarification and analysis.

Many of the operators were contacted directly, by telephone or visitation, to obtain additional information or to discuss reported conditions. In several instances where chemical data on the effluent discharges could not be found, the information was secured promptly on request to the person performing the analyses.

Data reviewed and recorded

It was determined that 49 municipal lagoons met the selection criteria. All available data on construction features, loadings and operating reports were reviewed. These consisted of (1) monthly operation reports for each lagoon system for the years 1972 and 1973 and (2) discharge data for the 2 years. These data are recorded on Table 1.

Loadings

Loadings shown in Table 1 for each lagoon system are expressed in terms of connected population for the lagoon capacities listed. Unfortunately there is insufficient flow data or raw sewage sampling for most lagoons to be of much value. It may be noted that most loadings are reasonably close to the design value of 100 persons per acre, although several are significantly lower than this and a few are well above. Discussion with basin engineers revealed that no industrial wastes are tributary to any of the systems either in significant volume or characteristics.

Lagoon capacity

Of the 49 systems studied, 27 have 2 cells, 19 have 3 cells, 2 have 4 and 1 has 5. The number of cells has no relationship to the total area or volume. Acreage apportionment between cells is frequently divided equally, although neither this ratio nor the relative depths of the cells follow any consistent pattern as may be noted from Table 1. Quite commonly maximum operating water depths range from 60 to 84 inches with 72 inches found most frequently.

Discharge patterns and periods

The most common pattern of discharge is to release the stored contents from the last cell in the series following a period of several days, usually about a week, with no inflow from preceding cells and to reverse the process after this discharge is completed. At some locations no isolation is provided and inflow from Cell No. 1 to Cell No. 2 (or to Cells 2 and 3) continues during the total drawdown period. Many communities do not record the information on Form D-4.9a in a sufficiently clear and complete fashion to show just what flow pattern is used and how it is changed during the period of discharge. This information was not required by the department until a year or so ago. All available information is recorded in Table 1, as explained in its attached legend.

Whenever possible the spring discharge is delayed until after the ice on the lagoon surface has completely melted. Some locations have found it necessary to discharge small quantities during the early spring ice cover when water levels reach elevations which would either produce automatic overflows or endanger the integrity of the dykes. From the column entitled No. of Days Since Ice-Out indicates that ice remained in 1972 on most lagoons until about April 15, but was gone by March 15 in 1973. Most systems are predicated on ice free conditions by April 1.

Periods of discharge shown under that title in Table 1 indicate a substantial variation in calendar period and number of days. Some of the periods shown include the intermediate isolation time between discharges from Cell No. 2 (or 2, 3, 4) and Cell No. 1; others clearly separate the two periods. The total length of discharge periods for the 186 discharges recorded in Table 1 are as follows:

No. Days Discharged	No. Discharge Periods
0 - 5	15
6 - 10	69
11 - 15	46
16 - 20	24
21 - 25	11
26 - 30	8
31 or more	13

MDPH
D-49

Exhibit 1

OPERATION REPORT OF WASTE STABILIZATION LAGOONS

____________________, Michigan

Month ____________ 19 ____ Operator ____________________

DATE	WEATHER					Flow MGD	Power KWH	LIQUID DEPTH-Ft.			D.O. Mg/l				% ICE COVER	ODORS [c]	REMARKS
	TEMP. °F		[a]	[b]	Precip. inches			CELL			TIME	CELL					
	Max.	Min.	Type	Wind				1	2	3		1	2	3			
1																	
2																	
3																	
4																	
5																	
6																	
7																	
8																	
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27																	
28																	
29																	
30																	
31																	
TOTAL																	

a. Weather Type – C- Clear CL - Cloudy R- Rain W- Windy CA- Calm
b. Wind – Indicate strength and direction – S- Strong M- Med. W- Weak
c. Odors – S- Slight M- Moderate O- Offensive C- Complaints

D-4.9 – INSTRUCTIONS

Weather – Report Daily

Temp. – Data should be from a reliable source in community. Data must be from a maximum-minimum thermometer.

Precip. (inches) – Data should be from a reliable source in the community, either at the plant or elsewhere. Do not use general area data. For information regarding official weather stations, write State Climatologist, Room 202, Manly Miles Building, 1405 South Harrison, East Lansing, Michigan 48823. If no reliable source is available install a precipitation gage as recommended by the State Climatologist.

Flow (MGD) – Report Daily

The raw sewage flow should be reported in million gallons per day (MGD), for example:

70,000 gallons per day as 0.07 MGD.
375,000 gallons per day as 0.38 MGD.
3,750,000 gallons per day as 3.75 MGD.

Use no more than three figures.

Power – Report Daily

Report the power consumed for pumping at the major pump station in kilowatt hours (KWH).

Liquid Depth – Report at Least Weekly

The depth to the nearest one-half foot in each of the lagoon cells, for example:

2 foot 2 inches as 2 ft.
4 foot 7 inches as 4½ ft.
5 foot 10 inches as 6 ft.

Dissolved Oxygen

Report time when a dissolved oxygen sample is taken. The dissolved oxygen concentration should be reported at least once a week in each cell during the period when the lagoon is not being discharged and once a day during discharge on the cell being discharged, recording the minimum D.O. value of at least three samples taken during the day (morning, noon and evening).

Ice Cover – Report at Least Weekly

Report the percentage of the lagoon system covered with ice. If measurements of ice and snow depths are taken report these results in remarks column.

Odors

Report odor condition as noted on form. If odors are not detectable indicate with dashed line in appropriate space. Discuss any unusual circumstances concerning odors in remarks column.

Remarks

This space should be reserved for any observations and information not included in other parts of this form. Observations on algae concentrations (both suspended and floating) color and turbidity in the lagoon along with comments on efforts to control rodents, insects and weeds should be reported. Problems with industrial waste, storm water or infiltration should also be noted. If additional space is necessary, please use Supplemental Remarks Sheet.

Flow Pattern

Indicate by brief sketches on Supplemental Remarks Sheet (MDPH – D-4.9a) the pattern (or route) of flow through the lagoon system.

Exhibit 2

D-4.9e

WASTE STABILIZATION LAGOONS
SUPPLEMENTAL REMARKS SHEET

____________________, Michigan

Month ____________________ 19 ____ Operator ________________________________

FLOW PATTERN (Sketch flow pattern in space provided below, indicating dates during month. If more than one pattern was used make an additional sketch for each period.)

From ____________________ To ____________________
(Date) (Date)

From ____________________ To ____________________
(Date) (Date)

From ____________________ To ____________________
(Date) (Date)

REMARKS

____

____

____

____

____

____

____

____

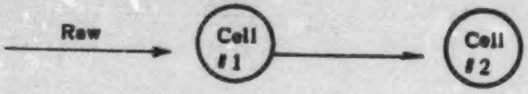
D-4.9a - INSTRUCTIONS

Flow Pattern

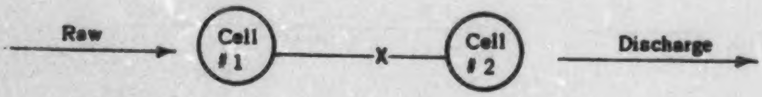
Indicate by brief sketches on Supplemental Remarks Sheet (MDPH - 4.9a) the pattern (or route) of flow through the lagoon system.

Examples:

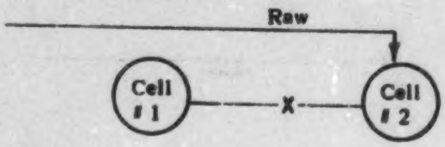
From October 1, 1968 (*Date*) To October 4, 1968 (*Date*)


Raw
Cell #1
Cell #2

From October 5, 1968 (*Date*) To October 13, 1968 (*Date*)


Raw
Cell #1
X
Cell #2
Discharge

From October 14, 1968 (*Date*) To October 22, 1968 (*Date*)


Raw
Cell #1
X
Cell #2

From October 23, 1968 (*Date*) To October 31, 1968 (*Date*)

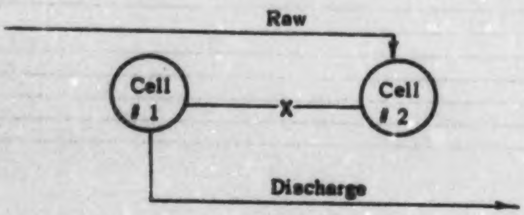
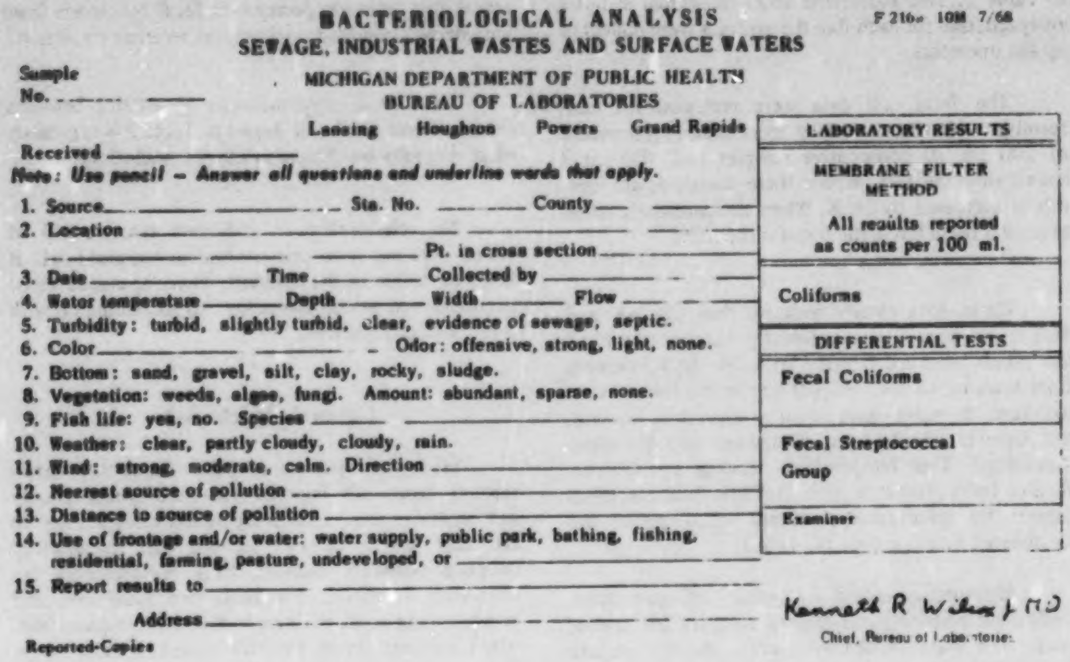

Raw
Cell #1
X
Cell #2
Discharge

Exhibit 3



F 216a 10M 7/68

BACTERIOLOGICAL ANALYSIS
SEWAGE, INDUSTRIAL WASTES AND SURFACE WATERS

MICHIGAN DEPARTMENT OF PUBLIC HEALTH
BUREAU OF LABORATORIES
Lansing Houghton Powers Grand Rapids

Sample No. ______

Received ______

Note: Use pencil – Answer all questions and underline words that apply.

1. Source ______ Sta. No. ______ County ______
2. Location ______ Pt. in cross section ______
3. Date ______ Time ______ Collected by ______
4. Water temperature ______ Depth ______ Width ______ Flow ______
5. Turbidity: turbid, slightly turbid, clear, evidence of sewage, septic.
6. Color ______ Odor: offensive, strong, light, none.
7. Bottom: sand, gravel, silt, clay, rocky, sludge.
8. Vegetation: weeds, algae, fungi. Amount: abundant, sparse, none.
9. Fish life: yes, no. Species ______
10. Weather: clear, partly cloudy, cloudy, rain.
11. Wind: strong, moderate, calm. Direction ______
12. Nearest source of pollution ______
13. Distance to source of pollution ______
14. Use of frontage and/or water: water supply, public park, bathing, fishing, residential, farming, pasture, undeveloped, or ______
15. Report results to ______

Address ______

Reported-Copies

LABORATORY RESULTS

MEMBRANE FILTER METHOD

All results reported as counts per 100 ml.

Coliforms

DIFFERENTIAL TESTS

Fecal Coliforms

Fecal Streptococcal Group

Examiner

Kenneth R Wilcox Jr MD
Chief, Bureau of Laboratories

Effluent quality–BOD and suspended solids

Records were reviewed and recorded for analyses of BOD and suspended solids on 1475 samples from the 49 lagoon systems. Each of these samples was a daily composite collected by the operator. This information is recorded under Discharge Data–Chemical in Table 1. Ranges of Values and arithmetic averages are shown for each seasonal discharge period.

The BOD and suspended solids data were analyzed statistically in terms of probability of occurrence. All values were arranged in order of magnitude and plotted on normal probability paper with concentration (mg/l) plotted against probability the value would not be exceeded under similar conditions. Separate plots were made for 2-cell and 3 or more cell systems. These plots (see Figures 1, 2, and 3) reveal the following quality levels.

% Probability of Occurrence	2 Cells		3 or More Cells	
Mean for Period	BOD	SS	BOD	SS
Most Probable	17	30	14	27
90% Probability	27	46	27	47

The distribution of values is reasonably consistent with normal distribution showing a high degree of uniformity for BOD and a reasonably good distribution for suspended solids. No statistically significant difference can be seen between the 2-cell and 3 or more cell systems for the loadings experienced.

The data do not reveal any detectable difference in levels of BOD or suspended solids between spring and fall discharges, including those from under the ice. Nor is any variation noted with variations in number of days discharged. Further, study of recorded water levels during discharge failed to show any definite trend related to rate of drawdown (inches per day) or from initial drawdown to low discharge level. The exception to this is when the water level is reduced below about 18 inches or when the point of drawoff is at or below that level. In these cases BOD and suspended solids concentration increase sharply.

Effluent quality–coliforms

Analyses for total coli and fecal coli performed by the bacteriological laboratories of the Michigan Department of Public Health were reported on standard forms F216a. A total of 1523 such forms were reviewed and the fecal coli per 100 mls recorded in Table 1. This represents 1523 individual samples analyzed, one for each day the samples were collected by the operators.

The fecal coli data were reviewed for conformity with the secondary treatment requirements of 200 for 30 consecutive samples and 400 for 7 consecutive samples. Where these standards are met, this is indicated by an X. When the geometric mean exceeded these levels the mean is recorded.

These data clearly indicate that the 200 and 400 levels were met consistently except: (1) When the ponds were ice covered or a few days following final thawing of ice, and (2) late in the fall at some locations as water temperatures reach low levels in the lagoons (Birch Run, Bridgman, and Plainfield Township). This temperature dependency deserves further study, but it is clear that low fecal coli levels cannot be achieved dependably when ponds are ice-covered or a few days afterward.

Although analyses for total coli have been performed routinely on lagoon samples for several years, the state regulatory agency did not require fecal coli data until about 2 years ago. Facts on fecal coli levels are just now emerging. There are still a few locations today where only total coli data are available as noted in Table 1.

An in-depth study of fecal coli data was made to examine seasonal trends in coli levels, both total and fecal. Typical facilities were selected and all available data are recorded in Table 2. These data reveal the dramatic decrease in fecal coli levels from ice covered conditions to warmer weather conditions.

The lack of any consistent relationship between total coli and fecal coli shown in Table 2 is typical of what generally was found at other installations.

No relationship or indicated trend could be found between fecal coli concentrations and levels of BOD or solids in the effluent. There appeared to be no difference in coli levels for 2 cells compared with 3 or more cell systems.

General Conclusions

Study of operational data and related information from 49 lagoons for two years clearly demonstrates that, with long period storage prior to fall and spring release, lagoons can consistently produce effluents meeting EPA requirements for secondary treatment for BOD and fecal coli and generally can meet the suspended solids requirement, when loadings are in the 100 person per acre range. No data were available to indicate the effect of higher loadings or short retention periods. It is recognized that performance at this level requires the application of generally accepted principles of design and construction and that a reasonable amount of attention by well trained and interested public works oriented operators is essential.

Exhibit 4

STATE OF MICHIGAN
DEPARTMENT OF NATURAL RESOURCES

WASTE STABILIZATION LAGOON DISCHARGE REPORT

R 4613 4/74

Month ______ 19 ______ Municipality ______ Michigan

Superintendent's Signature ______

Analyst ______

1 2 ID | 3–8 Plant No. | 9 10 Mo. | 11 12 Y | 13 15 Sampling Point Code

*	Fecal	Total
MF	31616	31504
MPN	31615	31505

D A Y	FLOW	BOD	SS	VSS	TOTAL-P	NH$_3$-N	pH	F. COLI*	T. COLI*	ICE COVER ON STREAM	REMARKS OR OTHER PARAMETERS									
	9	14	19	24	29	34	39	44	49	54	59	64	69	74 78	80	9	14	19	24	29
1 2	MGD	mg/l	mg/l	mg/l	mg/l	mg/l	SU	#/100ml	#/100ml	%					C					
PN	50050	00310	00530	00535	00665	00610	00400	31616 31615	31504 31505	01355					C					
SF															C					
01															C					
02															C					
03															C					
04															C					
05															C					
06															C					
07															C					
08															C					
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25															C					
26															C					
27															C					
28															C					
29															C					
30															C					
31															C					
TL															C					
ME															C					
WA															C					

WASTE STABILIZATION LAGOON DISCHARGE REPORT INSTRUCTIONS

Discharge reports are to be submitted by the tenth of the calendar month following a month during which a discharge occurred. They should be mailed to: Municipal Wastewater Division, Department of Natural Resources, Stevens T. Mason Building, Lansing, Michigan 48926.

Enter lagoon identification code number in the six spaces following the letters ID. In the next four spaces enter the date with the first two spaces being the month and the next two being the year. The last three spaces should contain the sampling point code assigned to your lagoon for final effluent. For example, ID 740055 10 73 000 would be the code for Yale's October 1973 discharge data.

If the results of a test reveal no detectable amount of the substance, then report that the amount is less than the least detectable amount. For example, if the least detectable amount of an analysis is 0.1 mg/l and the analysis indicates less than that amount, then report it as L 0.1. A dash should never be used. A blank implies that no analysis was made.

If the magnitude of the numbers for a particular parameter exceeds four digits, the row marked SF should be used. A Scaling Factor (SF) of 2 above a column would mean that the numbers reported in that column should be multiplied by 100 to equal the actual value of that parameter. Other Scaling Factors are as follows:

Multiplier	10	100	1000	10000	100000	Etc.
Scaling Factor	1	2	3	4	5	

IMPORTANT: In no instance shall any number be reported in more than four digits plus a decimal point or five digits without a decimal point.

FLOW - Total daily discharge flow should be reported using three figures. For example, 451600 gallons per day should be reported as 0.452 MGD.

REMARKS OR OTHER PARAMETERS - This space is provided for notes to the basin engineer or for the record. Other parameters may be reported in these columns by listing the parameter abbreviation and units in the top two rows, the Parameter Number (PN) in the third, and the Scaling Factor (SF), if any, in the fourth. Parameter Numbers may be obtained from your basin engineer or selected from the abbreviated list on the Additional Chemical Analyses Sheet (R4612).

TOTAL - Enter column totals in the row marked TL, where it is not blacked out.

MEAN - Enter the Arithmetic Mean (Average) for all parameters except Coliform. The Geometric Mean should be reported for the Coliform data.

If the discharge is covered by an NPDES permit with a 7 day Average (or Daily Maximum or Minimum) condition, then, for such parameters, report the required data in the row marked WA for Weekly Average. The 7 Day Average should be computed from what appears to be the worst seven consecutive days.

Table 1. Capacity and discharge data, waste stabilization lagoons, Michigan, 1972-73.

CAPACITY AND DISCHARGE DATA - WASTE STABILIZATION LAGOONS - MICHIGAN 1972-73

COMMUNITY	POPULATION		LAGOON CAPACITY							DISCHARGE DATA - EFFLUENT QUALITY													NOTES
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM-TO	# DAYS	# DAYS SAMPLING	BOD (mg/1) RANGE	BOD (mg/1) AVG	Susp. Sol. (mg/1) RANGE	Susp. Sol. (mg/1) AVG	# DAYS SAMPLING	RANGE	EPA STD. 200	EPA STD. 400	
Ashley	600	550	2	3.0	108	3.0	108				0	72-04-14 to 05-15	32	8	7-9	8	32-41	36	36	*≤10-36000	X	X	*Exceeded stds.before ice out
											10	73-03-23 to 03-30	8	8	9-15	11	11-63	24	8	≤10-320	X	X	
												73-04-23 to 04-27	5						5	≤10-20	X	X	
Belding	8000	4000	4	20.3	72	15.1	72	*15.0	72		0	72-04-11 to 04-27	17	9	2-15	5	14-60	30	9	≤10-1300	X	X	[a] Cells 3 & 4 each 7.5 acres
												72-10-02 to 10-25	24	11	1-2	1	2-14	9	11	≤10	X	X	
												73-03-26 to 04-09	24	9	1-11	7	27-58	35	chlorinated				
												73-11-03 to 11-13	9	3	3-10	6	no data		chlorinated				
Birch Run	1200	1300	2	6.0	72	6.0	96			A1	0	72-04-12 to 04-28	17	16	15-29	18	24-68	41	17	≤10-5100	X	775	Ice cover until 4-15-73
										B1	55	72-06-06 to 06-19	14	14	10-40	23	8-62	26	14	≤10-4100	X	X	2 Coli samples >210
										B1		72-10-28 to 10-30	8	7	12-20	15	10-31	16	7	≤10-350	X	X	
										A1	-	72-11-08 to 11-16	9	9	21-32	24	14-51	35	9	4600-5500	4900	5000	
										A4	30	73-04-12 to 05-07 10-22	25	18	19-38	27	36-140	50	18	≤10-750	X	X	
Breckenridge	1250	1800	2	6.0	72	6.0	72			B1	30	72-05-10 to 05-22	13	13	14-59	37	12-52	26	8	20-1100	X	X	No fecal coli data for other periods. Total coli tests.
										A1		72-10-16 to 10-31	16	10	6-26	21	10-80	43	No data				

Table 1. Continued.

	POPULATION		LAGOON CAPACITY							DISCHARGE DATA - EFFLUENT QUALITY													SHEET 2
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
															BOD (mg/1)		Susp. Sol. (mg/1)				EPA STD.		
COMMUNITY	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SIP'G ICE OUT	DATES FROM-TC	# DAYS	# DAYS SAMPLING	RANGE	AVG	RANGE	AVG	# DAYS SAMPLING	RANGE	200	400	NOTES
Breckenridge Continued	1250	1800	2	6.0	72	6.0	72			B1		72-11-13 to 11-24	12	5	3-26	17	21-67	45	No data				Continued No fecal col data for other period Total coli tests
										A1	30	73-04-12 to 04-27	16	8	12-140	81	11-102	43	No data				
										B1		73-05-23 to 05-31	9	3	40-60	47	27-64	45	No data				
										A1		73-10-24 to 10-31	8	5	15-23	18	58-78	60	No data				
Bridgman	1250	1308	3	4.5	72	4.0	72	4.0	72	A4	0	72-04-05 to 04-07	3	3	21-38	29	33-44	38	2	2100,530	1000	1000	Ice cover until 4-12
										A1	15	72-04-24 to 05-03	10	8	16-50	30	7-36	[illegible]	8	≤10-350	X	X	
										A2		72-05-04 to 05-12	7	5	18-28	23	13-24	17	4	≤10-860	X	X	
										A1		72-10-15 to 10-27	13	10	15-22	16	4-19	14	5	1300-4700	2500	2500	
										A1		72-11-12 to 11-21	10	8	16-26	17	10-31	17	6	640-4000	2500	2500	
										A4	30	73-04-18 to 05-20	32	15	15-48	30	6-65	28	19	≤10-600	X	X	
Brooklyn	1200	1107	3	3.0	78	3.5	72	3.5	72		40	72-05-15 72-05-25	11	7	11-27	18	28-68	30	8	≤10-10	X	X	
												72-10-17 to 10-26	10	4	5-10	7	4-16	10	8	≤10-120	X	X	
											25	73-04-09 to 04-18	10	6	10-29	20	27-122	77	7	≤10-140	X	X	
												73-10-12 to 10-18	7	4	4-5	5	3-31	14	5	≤10-500	X	X	
Brownstown Township Wayne County	1050	1360	2	7.7	60	10.8	60			A3		72-10-30 to 12-01	32	19	2-7	5	4-30	14	8	20-1800	X	X	
										A1	15	73-04-02 to 04-30	29	15	2-9	5	14-43	27	15	≤10-530	X	X	

Table 1. Continued.

SHEET 3

COMMUNITY	POPULATION		LAGOON CAPACITY							DISCHARGE DATA - EFFLUENT QUALITY													NOTES
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM-TO	# DAYS	# DAYS SAMPLING	BOD (mg/1) RANGE	BOD (mg/1) AVG	Susp. Sol. (mg/1) RANGE	Susp. Sol. (mg/1) AVG	# DAYS SAMPLING	RANGE	EPA STD. 200	EPA STD. 400	
Brownstown Township - Wayne County Continued	1050	1360	2	7.7	60	10.8	60					73-10-29 to 11-15	17	13	9-15	11	31-62	49	10	≤10-420	X	X	
Brown City	1850	1142	3	6.5	72	6.5	72	4.5	72		*0	72-04-10 to 04-24	15	11	8-35	25	15-62	35	11	≤30-130	X	X	* ice out about 4-15
											45	73-04-25 to 05-03	9	8	13-32	21	8-86	38	8	≤10-1300	X	X	
												73-10-19 to 11-02	11	8	5-14	8	5-28	15	7	≤10-340	X	X	
Byron Center											35	73-05-08 to 06-01	6	6	12-40	24	25-64	45	3	≤100	X	X	
Camden	600	405	2	3.3	69	2.6	60			B1		72-09-06 to 09-15	10	8 4	13-24 23-27	21 25	27-84 41-84	46 60	7	≤10-1300	X	X	
										A1		72-10-30 to 11-07	8	5	8-19	13	6-21	12	8	≤10-60	X	X	
										A2		72-11-17 to 11-21	5	5	14-22	18	20-25	22					
										A1	10	73-03-22 to 03-30	9	6	31-35	33	47-72	54	7	≤10-50	X	X	
												73-05-01 to 05-09	9	6	6-10	8	7-26	21	15	≤10-90	X	X	
												73-10-31 to 11-07	8	6	1-12	5	15-24	20	8	≤10-160	X	X	
Capac	1400	1279	2	7.0	72	7.0	72			A4	14	72-04-21 to 05-15	25	22	4-45	16	13-60	32	16	≤10-8000			
										A4		72-10-27 to 11-11	17	12	4-6	5	12-43	20	No data				Total coli only
										A4		72-11-20 to 11-30	8	6	5-7	6	22-46	29	No data				Total coli only

Table 1. Continued.

SHEET 4

COMMUNITY	POPULATION		LAGOON CAPACITY							DISCHARGE DATA - EFFLUENT QUALITY													NOTES
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COL.				
	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM TO	# DAYS	# DAYS SAMPLING	BOD (mg/1) RANGE	BOD (mg/1) AVG	Susp. Sol. (mg/1) RANGE	Susp. Sol. (mg/1) AVG	# DAYS SAMPLING	RANGE	EPA STD. 200	EPA STD. 400	
Capac Continued	1400	1279	2	7.0	72	7.0	72			A4	0	73-03-05 to 03-14	10	7	17-20	18	14-33	24	6	410-4800	1340	1340	Ice cover until 3-15
										A4	15	73-04-01 to 04-30	30	20	4-21	12	13-90	37	21	≤10-240	X	X	
										A4		73-10-17 to 11-09	23	17	2-10	5	3-59	30	17	≤10-800	X	X	
Carleton	2400	2314	5	5.0	72	4.5	72	4.5	72		15	72-04-26 to 05-05	10	4	11-34	22	32-81	62	7	≤10-140	X	X	
						Cell #4 4.4	84	Cell #5 5.0	84			72-05-10 to 06-01	22	3	10	10	7-41	20	19	≤10-20	X	X	
												72-11-23 to 12-12	20	6	8-25	15	24-48	39	12	10-770	X	X	
										A1	5	73-03-19 to 05-16	48	14	2-53	22	10-84	46	35	≤10-50	X	X	
												73-10-17 to 11-14	28	4	5-12	8	12-18	14	11	≤10-10	X	X	
Carson City	1500	1200	2	8.0	96	8.0	96			A1		72-11-06 to 11-16	11	11	14-52	25	12-33	24	11	≤10-10	X	X	
										A1	15	73-04-01 to 04-12	12	7	8-14	10	12-36	26	7	≤10-400	X	X	
Coleman	2000	1295	3	11.0	72	5.5	72	5.5	72	A1	15	72-05-01 to 05-05	5	5	32-75	51	10-50	32	5	≤10	X	X	
										A1		72-10-18 to 10-31	14	11	3-23	15	5-31	16	9	≤10-360	X	X	
										B1	25	73-04-11 to 04-30	20	11	15-34	21	16-92	49	8	≤10-330	X	X	
										A1		73-10-16 to 11-02	16	14	1-17	6	2-19	10	10	≤10-20	X	X	
Elkton			3							A		72 May 72 Oct							3	≤10	X	X	
										A1		73-05-02 to 05-08	7	3	16-21	18	51-90	68	No data				

Table 1. Continued.

	POPULATION		LAGOON CAPACITY							DISCHARGE DATA - EFFLUENT QUALITY													SHEET 5
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
COMMUNITY	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM-TO	# DAYS	# DAYS SAMPLING	BOD (mg/1) RANGE	BOD (mg/1) AVG	Susp. Sol. (mg/1) RANGE	Susp. Sol. (mg/1) AVG	# DAYS SAMPLING	RANGE	EPA STD. 200	EPA STD. 400	NOTES
Elkton Continued			3							A2		73-10-15 to 10-22	8	5	1-7	4	2-7	4	6	≤10-10	X	X	
Fennville	1800	810	3	10.0	72	4.0	72	4.0	72		0	72-04-05 to 04-11	7	5	11-16	13	21-41	34	5	≤10-180	X	X	
												72-06-13 to 06-22	10	8	6-30	16	17-80	33	7	≤10-400	X	X	
												72-11-14 to 12-01	17	13	7-40	15	13-59	40	5	≤10-800	X	X	
												73-04-06 to 04-15	8	6	11-16	13	36-86	55					
Fowler	1400	1000	2	8.0	78	8.0	10.2					72-11-13 to 11-17	5	5	4-7	6	10-21	15	5	≤10-30	X	X	
												72-12-07 to 12-14	8	4	8-10	9	10-22	14	4	≤10-40	X	X	
												73-04-05								≤10-170	X	X	
Fowlerville	2000	1990	3	6.0	72	6.0	72	6.0	72			72-09-06 to 09-11	6	4	2-6	3	3-14	10					
												72-11-06 to 11-21	16	4	4	4	2-5	3					
											20	73-04-05 to 04-17	13						8	≤10	X	X	
												73-11-06 to 11-14	9	8	3-8	5	5-11	7	6	≤10-890	X	X	
Hemlock	1400	1390	2	5.7	60	8.7	66			A4	0	72-03-26 to 04-07	10	7	18-54	44	6-16	10	7	2000-10000	4700	4700	Ice cover until 4-15
										A4	20	72-05-02 to 05-10	9	7	16-66	37	36-60	53	6	30-960	315	X	
										B1		72-06-01 to 06-06	6	4	32-44	38	22-34	26	4	≤10-80	X	X	
										A1		72-09-27 to 10-12	16	7	21-76	40	24-46	40	3	90-3900	340	X	
										B1		72-10-30 to 11-08	9	3	32-42	36	24-54	35	4	60-1500	240	240	

Table 1. Continued.

	POPULATION		LAGOON CAPACITY							DISCHARGE DATA EFFLUENT QUALITY													SHEET 6
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
															BOD (mg/l)		Susp. Sol. (mg/l)				EPA STD.		
COMMUNITY	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM-TO	# DAYS	# DAYS SAMPLING	RANGE	AVG	RANGE	AVG	# DAYS SAMPLING	RANGE	200	400	NOTES
Hemlock Continued	1400	1390	2	5.7	60	8.7	66			A1	20	73-04-05 to 04-18	14	9	18-36	26	6-32	20	12	180-4800	490	830	
										B1		73-04-30 to 05-11	12	9	18-32	23	20-40	25	9	≤10-50	X	X	
										B1		73-10-22 to 10-29	8	5	13-19	15	21-41	31	5	≤10-10	X	X	
Ithaca	3500	2420	3	15.1	72	9.8	72	9.6	72	A1	0	72-03-20 to 03-24	5	4	11-34	20	7-30	17	3	≤10-1380	X	X	Ice cover until 4-15
										A2	0	72-04-10 to 04-20	11	5	16-40	22	3-31	20	9	≤10-35000	500	1720	
										A3	24	72-05-08 to 05-23	16	6	12-52	28	22-86	50	11	≤10-190	X	X	
										A3		72-09-25 to 10-09	15	5	4-8	6	3-14	8	7	≤10-20	X	X	
										A1		72-11-20 to 11-29	10	4	16-19	17	3-40	16	6	≤10	X	X	
										A3	15	73-03-31 to 04-26	17	6	12-70	40	25-70	45	14	≤10-120	X	X	
										A3		73-05-21 to 06-01 Oct 8	12	3	20-40	27	3-31	19	3	≤10-350	X	X	
Kent City	900	700	3	4.5	60	2.0	60	2.0	84	A1	15	72-04-27 to 05-04	8	7	16-37	14	34-64	43	10	≤10-300	X	X	
										A1		72-10-30 to 11-11	19	17	3-15	8	16-40	26	16	≤10-1100	X	X	
										A1	10	73-03-26 to 03-30	5	4	27-32	28	No data		6	≤10-50	X	X	
										A1		73-04-11 to 04-14	4	3	17-23	19	No data		2	≤10	X	X	
										A1		73-05-21 to 05-24	4	3	7-9	8	14-18	17	3	≤10	X	X	
										A1		73-11-04 to 11-12	3	7	7-11	9	No data						

Table 1. Continued.

SHEET 7

	POPULATION			LAGOON CAPACITY						DISCHARGE DATA - EFFLUENT QUALITY													
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
COMMUNITY	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM-TO	# DAYS	# DAYS SAMPLING	BOD (mg/1) RANGE	AVG	Susp. Sol. (mg/1) RANGE	AVG	# DAYS SAMPLING	RANGE	EPA STD. 200	400	NOTES
Laingsburg	1300	1100	3	6.5	96	6.5	96	*18.3	96	B3	13	72-04-18 to 04-24	7	5	22-40	30	48-62	51	5	≤10	X	X	*Includes storage
										A3		72-11-06 to 11-31	17	10	2-26	12	4-26	11	6	≤10	X	X	Flow storage
										A4	10	73-03-26 to 04-30	35	19	2-20	8	1-20	14	22	≤10-1200	X	X	
										A1		73-11-04 to 11-12	9	7	7-11	9	No data		3	≤10-10	X	X	
Lakeview	1150	1100	3	3.7	72	4.5	78	3.3	74	A4	10	*72-04-24 to 05-05	12	8	29-50	41	42-58	52	10	130-5300	675	1100	*Drawoff pipe broken. Discharge from bottom of lagoon
										A3		72-11-06 to 11-31	25	17	2-26	12	4-26	11	12	≤10-6100	X	X	
										A3	30	73-04-14 to 05-04	20	9	11-40	23	3-60	23	11	≤10-300	X	X	
										A4		73-11-12 to 11-22	11	6	6-15	11	3-13	9					
Maybee	1050	485	3	2.5	78	2.5	78	46	78	B1	10	73-03-22 to 03-30	9	8	17-29	24	28-42	33	10	≤10-340	X	X	
										A1		73-04-02 to 05-05	33	5	22-26	24	6-13	11	19	≤10-80	X	X	
												73-10-23 to 11-01							9	≤10-30	X	X	
Memphis	2000	1120	2	8.0	72	12.8	72			A1	30	73-04-14 to 04-23	10		No data		No data		6	≤10-20	X	X	
										B1		73-05-05 to 05-18	14	6	21-28	24	36-94	66	7	≤10-320	X	X	
										A1		73-10-22 to 10-29	8						5	60-330	X	X	
Millington	1800	1500	2	8.8	72	12.8	72			A1		72-10-02 to 10-12	11		3-5	6			5	≤10-710	X	X	
											25	73-04-09 to 04-18	10				2-33	12	8	≤10-380	X	X	
												73-10-22							6	≤10	X	X	

Table 1. Continued.

SHEET 8

	POPULATION		LAGOON CAPACITY							DISCHARGE DATA - EFFLUENT QUALITY													
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
COMMUNITY	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM-TO	# DAYS	# DAYS SAMPLING	BOD (mg/1) RANGE	BOD (mg/1) AVG	Susp. Sol. (mg/1) RANGE	Susp. Sol. (mg/1) AVG	# DAYS SAMPLING	RANGE	EPA STD. 200	EPA STD. 400	NOTES
Morenci	2850	2200	2	14.8	84	13.9	84			A1	0	72-03-27 to 04-08	12	10	2-36	20	3-27	12	10	≤10-20	X	X	
												72-05-02 to 05-16	15	10	9-32	23	10-69	27	10	≤10	X	X	
												72-10-02 to 10-13	12	10	1-6	4	1-9	4	5	≤10-10	X	X	
												72-11-03 to 11-10	6	6	5-9	6	32-80	50	6	≤10-800	X	X	
											10	73-03-22 to 04-06	15	12	5-15	8	2-18	12	12	≤10-20	X	X	
												73-04-23 to 05-04	12	9	7-22	13	5-73	25	11	≤10-190	X	X	
										A1		73-10-08 to 11-08	30						15	≤10-400	X	X	
Mulliken	750	450	3	2.5	60	2.5	72	2.5	84	A3		73-04-13 to 05-23	40	18	3-9	5	3-84	27	18	≤10-40	X	X	
North Branch	1300	1100	3	8.9	72	4.3	72	4.3	72	A2	1	72-0[illegible]-12 to 04-21	10	7	23-47	34	36-101	61	6	≤10-630	X	X	
												72-10-09 to 10-13	5	5	2-9	6	2-15	7	5	≤10-40	X	X	
											20	73-04-06 to 04-24	9	7	7-36	18	5-39	18	7	≤10	X	X	
												73-11-05 to 11-16	12	8	4-13	8	7-35	19	4	≤10	X	X	
Ovid	1600	1870	3	4.5	72	4.5	72	7.0	78	A1	10	72-04-24 to 05-15	22						9	≤10-[illegible]00	X	X	
										A1		72-11-01 to 11-17	9	8	6-15	8	12-57	29	8	≤10-30	X	X	
										A1	15	73-03-28 to 04-08	11	8	7-18	11	9-60	32	9	≤10-160	X	X	

Table 1. Continued.

	POPULATION		LAGOON CAPACITY							DISCHARGE DATA - EFFLUENT QUALITY													SHEET 9
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
COMMUNITY	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM-TO	# DAYS	# DAYS SAMPLING	BOD (mg/1) RANGE	BOD (mg/1) AVG	Susp. Sol. (mg/1) RANGE	Susp. Sol. (mg/1) AVG	# DAYS SAMPLING	RANGE	EPA STD. 200	EPA STD. 400	NOTES
Ovid Continued	1600	1870	3	4.5	72	4.5	72	7.0	78			73-04-20 to 04-30	11	5	5-17	12	6-26	15	5	≤10-10	X	X	
												73-10-30 to 11-05	7						6	≤10	X	X	
Pentwater	1500	1385	4	6.9	48	4.2	48	4.2	72	A2	10	72-04-24 to 05-05	13	10	7-18	12	20-36	27	11	≤10-70	X	X	
												72-10-21 to 12-06	46	31	14-40	25	20-58	35	18	*760-7000	2630	3700	*Leaky valve permitted discharge of primary cell into effluent line.
											15	73-04-02 to 05-04	33	25	9-42	20	12-54	24	25	≤10-310	X	X	
																			16	≤10-60	X	X	
Pigeon	1300	1175	2	6.5	72	6.5	96			A4		72-10-09 to 10-20	12	10	5-34	21	3-74	43	10	≤10-820	X	X	
										A4	5	73-03-20 to 04-10	21						8	30-1400	X	425	
										A4		73-10-16 to 11-05	16	11	9-17	14	8-71	25	13	50-13800	730	1020	
Plainfield Township	370	300	2	2.0	60	1.7	60			A4		72-11-22 to 12-07	15	6	4-18	9	35-45	39	6	930-3300	2100	2100	
										A4	20	73-04-06 to 04-19	14	6	18-34	25	22-60	40	10	190-11700	2000	2100	
Port Sanilac Summer Pop.	700 1200	500 900	3	4.7	72	5.6	72	5.8	96	A3	30	73-04-23 to 05-07	15	9	6-28	10	8-54	25	10	≤10-100	X	X	
Potterville	1400	1360	2	7.0	60	7.0	72				15	72-05-01 to 05-09	7	7	21-46	32	52-144	79	7	10-340	X	X	
											40	73-04-23 to 04-30	8	8	12-19	15	34-66	51	8	10-390	X	X	

Table 1. Continued.

	POPULATION		LAGOON CAPACITY							DISCHARGE DATA - EFFLUENT QUALITY													SHEET 10
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
															BOD (mg/1)		Susp. Sol. (mg/1)				EPA STD.		
COMMUNITY	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM-TO	# DAYS	# DAYS SAMPLING	RANGE	AVG	RANGE	AVG	# DAYS SAMPLING	RANGE	200	400	NOTES
Reading	1250	1120	2	7.6	72	7.9	72			B2		72-05-16 to 05-30	15	3	8-23	16	27-52	37	9	≤10-20	X	X	
										B2		72-10-16 to 11-01	16	5	7-36	15	1-25	13	12	≤10-30	X	X	
										A1	10	73-03-22 to 04-02	12	7	24-34	30	27-53	42	6	≤10-700	X	X	
										B1		73-04-26 to 05-08	13	9	2-19	8	5-35	13	9	≤10-20	X	X	
										A1		73-10-30 to 11-02	4	4	15-18	16	18-28	23	4	≤10-80	X	X	
Reese	1450	1200	2	7.2	72	7.2	96					73-04-12 to 04-26	11	11	2-18	10	66-227	119	9	≤10-160	X	X	
												73-10-30 to 11-06							8	20-530	X	X	
Saint Charles	2400	1950	2	11.0	60	13.2	60			B1		72-10-02 to 10-09	8	5	2-3	3	7-32	15	3	≤ 10	X	X	
										A1		72-11-13 to 11-18	6	5	6-17	10	4-12	6	4	≤ 10-310	X	X	
											15	73-03-29 to 04-08	11	5	13-21	16	20-40	27	8	≤ 10-240	X	X	
												73-05-02 to 05-09	8	6	21-54	34	46-50	55	6	≤10-10	X	X	
										A1		73-10-29 to 11-08							7	≤10-100	X	X	
Sand Lake	448	365	2	2.0	96	2.0	96			A1	15	72-04-26 to 05-01	7	6	[illegible]-46	23	25-61	39	6	≤10-600	X	X	
										A1	25	73-04-09 to 04-17	9	6	18-29	23	56-104	79	6	20-700	X	X	
												73-11-06 to 11-12	7	4	6-9	8	3-21	19					

Table 1. Continued.

	POPULATION		LAGOON CAPACITY							DISCHARGE DATA - EFFLUENT QUALITY													SHEET 11
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
COMMUNITY	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM-TO	# DAYS	# DAYS SAMPLING	BOD (mg/1) RANGE	BOD (mg/1) AVG	Susp. Sol. (mg/1) RANGE	Susp. Sol. (mg/1) AVG	# DAYS SAMPLING	RANGE	EPA STD. 200	EPA STD. 400	NOTES
Scottville	1900	1140	2	8.9	120	10.1	120			A3	10	72-04-28 to 05-19	8	5	37-60	48	17-74	54	15	≤10-130	X	X	
										A4		72-11-01 to 11-28	28	19	6-35	23	3-38	21	19	≤10-3400	X	800	
										A1	15	73-04-01 to 06-07	68	39	11-80	37	7-170	25	6	≤10	X	X	
Shepherd	1500	1460	2	10.6	72	6.1	72			A4	0	†72-03-28 to 04-19	22	15	11-29	19	12-72	44	13	290-31500	2950	7200	†Ponds covered with ice nearly all this period
										A4	15	72-04-29 to 05-25	36	23	10-50	23	6-68	32	20	≤10-1870	X	X	
										A4		72-11-01 to 11-30	30	21	5-44	18	2-82	16	21	≤10-760	X	X	
										A4		73-10-15 to 10-31	17	8	13-30	23	10-40	19	5	≤10-40	X	X	
Stanton	500	400	2	4.0	84	3.0	96				15	72-05-01 to 05-08	8	5	7-28	19	11-33	19	6	≤10-100	X	X	
												72-10-23 to 10-27	5	6	7-28	18	11-23	15	9	≤10-70	X	X	
												73-03-29 to 04-16	19	8	8-23	17	12-36	30	4	≤10-10	X	X	
Suttons Bay	700	500	3	2.5	54	2.5	60	2.0	60	A1	1	72-04-22 to 05-22	30	15	10-15	13	12-62	34	20	≤10-100	X	X	
										A1		72-09-12 to 09-20	9	4	5-9	7	12-20	14	4	≤10-210	X	X	
												72-11-20 to 12-18	9	9	6-10	8	3-37	12					
												10-03 to 11-06							6	≤10-40			

Table 1. Continued.

	POPULATION		LAGOON CAPACITY							DISCHARGE DATA - EFFLUENT QUALITY													SHEET 12
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
															BOD (mg/1)		Susp. Sol. (mg/1)				EPA STD.		
COMMUNITY	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM-TO	# DAYS	# DAYS SAMPLING	RANGE	AVG	RANGE	AVG	# DAYS SAMPLING	RANGE	200	400	NOTES
Vernon	900	815	3	2.8	72	3.0	72	3.3	72	A4		72-10-18 to 10-30	13	9	2-10	4	6-22	10	8	≤10-20	X	X	
										A2	10	73-03-26 to 04-11	16	11	13-25	19	22-102	49	9	≤10-220	X	X	
Waldron	600	525	2	3.1	72	3.1	72					72-05-05 to 05-10	6	4	6-24	15	72-88	80	4	≤10	X	X	
												72-09-25 to 10-17	23	16	5-37	20	12-70	33	13	≤10-6300	X	X	
										A4		73-10-07 to 11											
Wheatland Township	1025	800	3	3.9	72	3.0	72	3.0	96		5	72-04-24 to 05-11	18	10	25-38	30	72-120	101	10	≤10-180	X	X	
												72-10-31 to 11-27	28						19	≤10-10	X	X	
												73-04-01 to 05-04	34	24	9-18	15	6-50	42	11	≤10-90	X	X	
												73-10-08 to 11-01	24	19	1-4	2	5-56	20	10	≤10-680	X	X	
Wright Township Ottawa County												72-04-10 to 04-28	19	6	10-25	17	37-92	65					
												72-11-08 to 11-13	6	5	4-6	5	3-30	15					
												72-12-11 to 12-13	3	3	1-5	3	10-38	20					
												73-04-08 to 04-11	4	3	17-45	29	42-52	35	3	≤10-80	X	X	

Table 1. Continued.

SHEET
13

	POPULATION		LAGOON CAPACITY							DISCHARGE DATA - EFFLUENT QUALITY													
				CELL 1		CELL 2		CELL 3				PERIOD OF DISCHARGE		CHEMICAL					FECAL COLI				
COMMUNITY	DESIGN	CONN	# CELLS	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	AREA (AC)	DEPTH (INS)	FLOW PATTERN	# DAYS SINCE ICE OUT	DATES FROM-TO	# DAYS	# DAYS SAMPLING	BOD (mg/l) RANGE	AVG	Susp. Sol. (mg/l) RANGE	AVG	# DAYS SAMPLING	RANGE	EPA STD. 200	400	NOTES
Yale	1800	1505	2	11.5	72	6.5	72				5	72-04-17 to 04-28	12	10	16-53	24	38-109	46	10	∠10-1140	X	X	
												72-05-06 to 05-20	15	10	3-6	4	10-25	19	9	∠10-720	X	X	
											30	73-04-16 to 04-27	12	10	5-11	7	14-76	33	10	∠10-30	X	X	
												73-05-07 to 05-18	12	10	5-12	8	20-46	28	10	∠10-10	X	X	
												73-10-22 to 11-02	11	10	2-3	3	11-40	19	10	∠10-20	X	X	
Coopersville	3600	2400	3	10.0	72	10.0	72	*12.0	*96	A1		72-11-01 to 11-15	16	11	2-10	4	4-56	28	No data				*Total for cells 3 & 4
												73-05-08 to 05-15	8	4	2-7	5	14-30	20	5	∠10	X	X	

Table 2. Typical data showing seasonal trends in coli concentrations, lagoon effluents, Michigan.

COMMUNITY	SAMPLE NO.	DATE	TOTAL COLI	FECAL	COMMENTS
Ashley (Spring 1972 Discharge)	1	4-10	120,000	2,600	
	2	4-11	322,000	3,010	
	3	4-12	265,000	2,750	
	4	4-13	355,000	3,000	
	5	4-14	450,000	12,100	
	6	4-15	51,000	900	
	7	4-16	23,000,000	36,000	
	8	4-17	290,000	810	
	9	4-18	117,000	1,160	
	10	4-19	1,230,000	1,030	Ponds clear of ice on April 19th.
	11	4-20	66,000	490	
	12	4-21	83,000	400	
	13	4-22	73,000	320	
	14	4-23	68,000	570	
	15	4-24	78,000	360	
	16	4-25	67,000	950	
	17	4-26	8,100	90	
	18	4-27	31,000	20	
	19	4-28	1,600	<10	
	20	4-29	400	<10	
	21	4-30	9,000	<10	
	22	5-1	500,000	<10	
	23	5-2	11,600	10	
	24	5-3	68,000	50	
	25	5-4	46,000	130	
	26	5-5	55,000	630	
	27	5-6	540,000	3,700	
	28	5-7	74,600	430	
	29	5-8	300,000	3,100	
	30	5-9	48,000	180	30 day geometric mean equals 300. Quality meets EPA standard for period April 20 to May 15 after ice cover had thawed.
	31	5-10	28,000	10	
	32	5-11	10,500	<10	
	33	5-12	6,900	<10	
	34	5-13	6,300	30	
	35	5-14	10,800	<10	
	36	5-15	65,000	<10	
Ashley (Spring 1973 Discharge)	—	2-28	660,000	6,300	Preliminary - No discharge. Ponds clear of ice 3-15.
	1	3-23	1,600	<10	
	2	3-24	21,000	170	
	3	3-25	22,000	90	
	4	3-26	23,000	120	
	5	3-27	56,000	130	
	6	3-28	4,800	220	
	7	3-29	47,000	130	
	8	3-30	31,000	320	
	9	4-23	—	<10	
	10	4-24	—	10	
	11	4-25	—	<10	
	12	4-26	—	20	
	13	4-27	—	<10	
Capac (Spring 1972 Discharge)	1	3-28	12,400,000	1,210	
	2	3-29	645,000	210	
	3	3-30	150,000	40	
	4	3-31	865,000	830	
	5	4-3	1,200,000	3,300	
	6	4-4	1,800,000	1,000	
	7	4-5	91,000 000	2,100	
	8	4-6	860,000	3,500	
	9	4-7	1,850,000	630	
	10	4-10	496,000	30	
	11	4-11	52,300,000	20,200	
	12	4-12	35,000,000	1,500	

Table 2. Continued.

COMMUNITY	SAMPLE NO.	DATE	TOTAL COLI	FECAL	COMMENTS
Capac Continued (Spring 1972 Discharge)	13	4-13	3,000,000	3,000	
	14	4-14	9,180,000	2,800	
	15	4-17	630,000	790	
	16	4-18	35,000	780	
	17	4-19	5,100,000	4,000	
	18	4-20	900,000	7,000	
	19	4-21	620,000	1,150	
	20	4-24	26,000	450	
	21	4-25	29,000	400	
	22	4-26	52,000	850	
	23	4-27	54,000,000	1,900	
	24	4-28	118,000	160	
	25	5-01	65,000	170	
	26	5-02	39,000	80	
	27	5-03	34,000	30	
Capac (Spring 1973 Discharge)	1	4-02	3,200	10	
	2	4-03	17,000	10	
	3	4-04	33,000	20	
	4	4-05	3,200	50	
	5	4-06	30,000	240	
	6	4-09	14,200	20	
	7	4-10	37,000	50	
	8	4-11	11,000	30	
	9	4-12	420,000	10	
	10	4-13	9,300	170	
	11	4-16	6,700	10	
	12	4-17	5,600	100	
	13	4-18	3,900	10	
	14	4-19	7,500	50	
	15	4-20	54,000	40	
	16	4-23	22,000	10	
	17	4-24	4,500	10	
	18	4-25	420,000	90	
	19	4-26	260,000	50	
	20	4-27	18,000	140	
	21	4-30	73,000	150	
Ovid (Winter-Spring 1972)	1	3-14	2,900,000	30,000	Laggons were full on March 14 and overflowing under iced-over conditions until about April 15. Controlled discharge began under ice-free conditions on April 24.
	2	3-15	1,900,000	1,000	
	3	3-16	2,300,000	50,000	
	4	3-17	7,050,000	53,200	
	5	3-18	5,100,000	53,500	
	6	3-20	377,000	2,800	
	7	3-21	595,000	20,500	
	8	3-22	460,000	33,000	
	9	3-23	590,000	42,000	
	10	3-24	37,800	2,160	
	11	3-27	2,300,000	1,580	
	12	3-28	680,000	2,610	
	13	3-29	540,000	100	
Ovid	1	4-24	12,000	400	
	2	4-25	22,000	40	
	3	4-26	41,000	90	
	4	4-27	30,000	40	
	5	4-28	42,000	250	
	6	4-29	94,000	390	
	7	5-13	720,000	<10	
	8	5-15	28,000,000	170	
Ovid (Fall 1972 Discharge)	1	10-03	9,300	<10	Discharged from cell No. 3 under controlled conditions following several months' storage.
	2	11-01	400	<10	
	3	11-09	2,900	<10	
	4	11-10	800	<10	
	5	11-11	2,000	<10	

6

Table 2. Continued.

COMMUNITY	SAMPLE NO.	DATE	TOTAL COLI	FECAL	COMMENTS
Ovid Continued (Fall 1972 Discharge)	6	11-12	1,900	<10	
	7	11-13	1,400	<10	
	8	11-14	6,700	<10	
	9	11-15	22,000	30	
	10	11-16	4,400	10	
	11	11-17	19,000	<10	
Ovid (Winter-Spring 1973 Overflow)	1	1-16	1,300	<10	
	2	1-17	1,800	<10	
	3	1-18	1,900	<10	
	4	1-19	1,600	<10	
	5	1-20	5,700	90	
	6	1-21	5,000	110	
	7	1-22	5,200	30	
	8	3-12	330,000	4,100	
	9	3-13	230,000	3,100	
	10	3-14	360,000	4,800	
	11	3-15	43,000	1,600	Ice cover nearly off on March 12.
	12	3-28	1,000	<10	
	13	3-29	100	<10	
	14	3-30	31,000	<10	
	15	3-31	110,000	<10	
	16	4-02	4,300	10	
	17	4-03	74,000	40	
	18	4-04	6,300	160	
	19	4-05	76,000	70	
	20	4-06	42,000	<10	
	1	4-23	42,000	<10	
	2	4-24	45,000	10	
	3	4-25	24,000	<10	
	4	4-26	1,500	<10	
	5	4-27	2,000	10	
Rose City (Winter-Spring 1972 Overflow & Discharge)	1	3-06	4,100,000	60,000	
	2	3-07	4,900,000	110,000	
	3	3-08	2,100,000	10,000	
	4	3-09	1,300,000	10,000	
	5	4-08	800,000	16,000	
	6	4-09	400,000	12,000	
	7	4-10	1,700,000	47,000	
	8	4-11	11,200,000	31,000	Ponds still ice-covered.
	9	5-06	1,700,000	3,300	Cell No. 1.
Rose City (Fall 1972 Discharge)	1	9-27	50,000	10	
	2	9-28	480,000	290	
	3	10-17	47,000	50	
	4	10-18	2,000	10	
	5	10-19	3,000	10	
	6	10-20	4,200	30	
	7	10-21	1,800	<10	
	8	11-08	230,000	385	Cell No. 1
	9	11-09	210,000	3,700	Cell No. 1
	10	11-10	48,000	1,800	Cell No. 1
Rose City (Winter 1972-73 Overflow)	11	12-19	4,300	<10	
	12	12-20	2,500	<10	
	13	12-22	1,800	<10	
	14	1-25	35,000	4,300	Ponds covered with ice.
	15	1-26	240,000	2,000	
	16	1-27	600,000	30,000	
	17	1-28	23,000	2,900	
	18	1-30	5,700,000	9,100	
	19	1-31	2,200,000	24,000	
	20	2-01	6,000,000	7,200	
	21	2-02	2,300,000	33,000	
	22	3-19	[illegible]8,000,000	3,400	
	23	3-20	380,000	345	Ice about out.

Table 2. Continued.

COMMUNITY	SAMPLE NO.	DATE	TOTAL COLI	FECAL	COMMENTS
Rose City (Spring 1973 Discharge)	1	4-09	480,000	10	
	2	4-10	34,000	50	
	3	4-11	790,000	150	
	4	4-17	120,000	≤10	
	5	4-18	3,400	≤10	
	6	4-19	8,000	20	
	7	4-23	2,000	≤10	
	8	4-24	4,000	≤10	
Pentwater (Spring 1972 Discharge)	1	4-24	6,100	10	
	2	4-24	8,000	≤10	
	3	4-25	131,000	≤10	
	4	4-26	97,000	≤10	
	5	4-27	36,000	≤10	
	6	4-28	146,000	≤10	
	7	5-01	86,000	≤10	
	8	5-02	190,000	≤10	
	9	5-03	89,000	10	
	10	5-04	310,000	≤10	
	11	5-05	105,000	70	
Pentwater (Spring 1973 Discharge)	1	4-02	800	≤10	
	2	4-03	2,000	≤10	
	3	4-04	3,200	10	
	4	4-05	1,300	≤10	
	5	4-06	40,000	≤10	
	6	4-09	36,000	≤10	
	7	4-10	2,100	≤10	
	8	4-11	300	≤10	
	9	4-12	26,000	≤10	
	10	4-13	52,000	60	
	11	4-16	34,000	20	
	12	4-17	17,000	10	
	13	4-18	23,000	≤10	
	14	4-19	400	≤10	
	15	4-20	1,900	≤10	
	16	4-23	3,900	20	
	17	4-24	29,000	10	
	18	4-25	38,000	310	
	19	4-26	3,500	≤10	
	20	4-27	5,800	≤10	
	21	4-30	900	≤10	
	22	5-01	2,300	≤10	
	23	5-02	24,000	20	
	24	5-03	6,600	≤10	
	25	5-04	5,200	20	
Scottville (Spring 1972 Discharge)	1	5-01	37,000	≤10	
	2	5-02	38,000	30	
	3	5-03	52,000	30	
	4	5-04	213,000	10	
	5	5-05	38,000	130	
	6	5-08	32,000	70	
	7	5-09	113,000	70	
	8	5-10	2,500	≤10	
	9	5-11	780,000	≤10	
	10	5-12	9,300,000	≤10	
	11	5-15	43,000	≤10	
	12	5-16	83,000	≤10	
	13	5-17	42,000	≤10	
	14	5-18	14,200	≤10	
	15	5-19	2,000	≤10	

Table 2. Continued.

COMMUNITY	SAMPLE NO.	DATE	TOTAL COLI	FECAL	COMMENTS
Scottville (Fall 1972 Discharge)	1	11-01	6,500	60	
	2	11-02	9,600	20	
	3	11-03	26,000	160	
	4	11-06	190,000	20	
	5	11-07	22,000	200	
	6	11-08	9,000	<10	
	7	11-09	12,000	30	
	8	11-10	140,000	580	
	9	11-13	34,000	230	
	10	11-14	230,000	1,100	
	11	11-15	230,000	710	
	12	11-16	41,000	——	
	13	11-17	210,000	3,500	
	14	11-20	21,000	70	
	15	11-21	100,000	2,001	
	16	11-22	53,000	90	
	17	11-24	90,000	680	
	18	11-27	370,000	970	
	19	11-28	400,000	2,700	
Scottville (Spring 1973 Discharge)	1	4-16	25,000	10	
	2	4-17	19,000	<10	
	3	4-18	2,300	<10	
	4	4-19	25,000	<10	
	5	4-23	37,000	<10	
	6	4-24	3,700	<10	
Shepherd (Spring 1972 Discharge)	1	3-29	1,100,000	7,000	
	2	3-30	1,400,000	10,000	
	3	4-04	730,000	6,000	
	4	4-05	764,000	18,600	
	5	4-06	625,000	31,500	
	6	4-10	490,000	4,200	
	7	4-11	7,300,000	1,100	
	8	4-12	126,000	390	
	9	4-13	94,500	990	
	10	4-17	77,000	290	
	11	4-18	106,000	2,390	
	12	4-19	33,000	490	
	13	4-20	31,000	870	
	14	4-24	40,000	650	
	15	4-25	54,500	390	
	16	4-26	20,000	150	
	17	4-27	9,000	30	
	18	5-01	36,500	40	
	19	5-02	250,000	1,870	
	20	5-03	41,000	560	
	21	5-04	14,900	130	
	22	5-08	65,000	170	
	23	5-09	89,000	1,190	
	24	5-10	250,000	110	
	25	5-11	8,500	10	
	76	5-15	71,000,000	<10	
	27	5-16	8,000,000	<10	
	28	5-17	11,600,000	<10	
	29	5-18	9,800	<10	
	30	5-22	96,000	<10	
	31	5-23	98,000	10	
	32	5-24	4,700,000	40	
	33	5-25	51,000	20	

Table 2. Continued.

COMMUNITY	SAMPLE NO.	DATE	TOTAL COLI	FECAL	COMMENTS
Shepherd (Fall 1972 Discharge)	1	10-30	23,000	10	
	2	10-31	1,600	≤10	
	3	11-01	2,100	10	
	4	11-02	1,000	≤10	
	5	11-06	2,100	≤10	
	6	11-07	2,200	40	
	7	11-08	3,300	40	
	8	11-09	2,300	20	
	9	11-13	2,000	30	
	10	11-14	1,500	≤10	
	11	11-15	3,000	30	
	12	11-16	3,400	20	
	13	11-20	300	10	
	14	11-21	2,800	≤10	
	15	11-22	32,000	30	
	16	11-27	20,000	760	
	17	11-28	8,600	370	
	18	11-29	15,000	350	
	19	11-30	1,900	≤10	

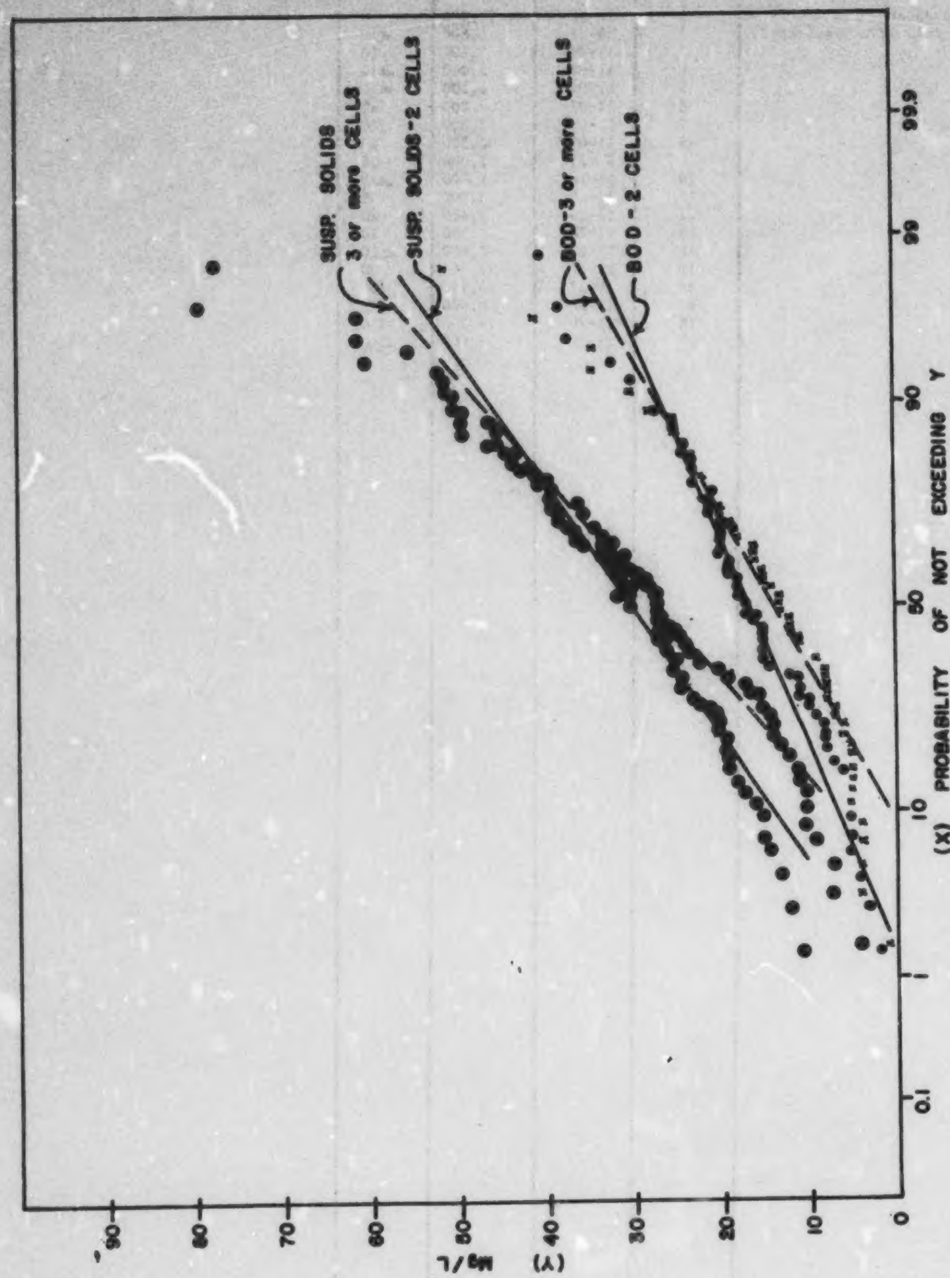

Figure 1. Comparison of effluent quality 2-cell systems vs 3 or more with long period storage before discharge, Michigan.

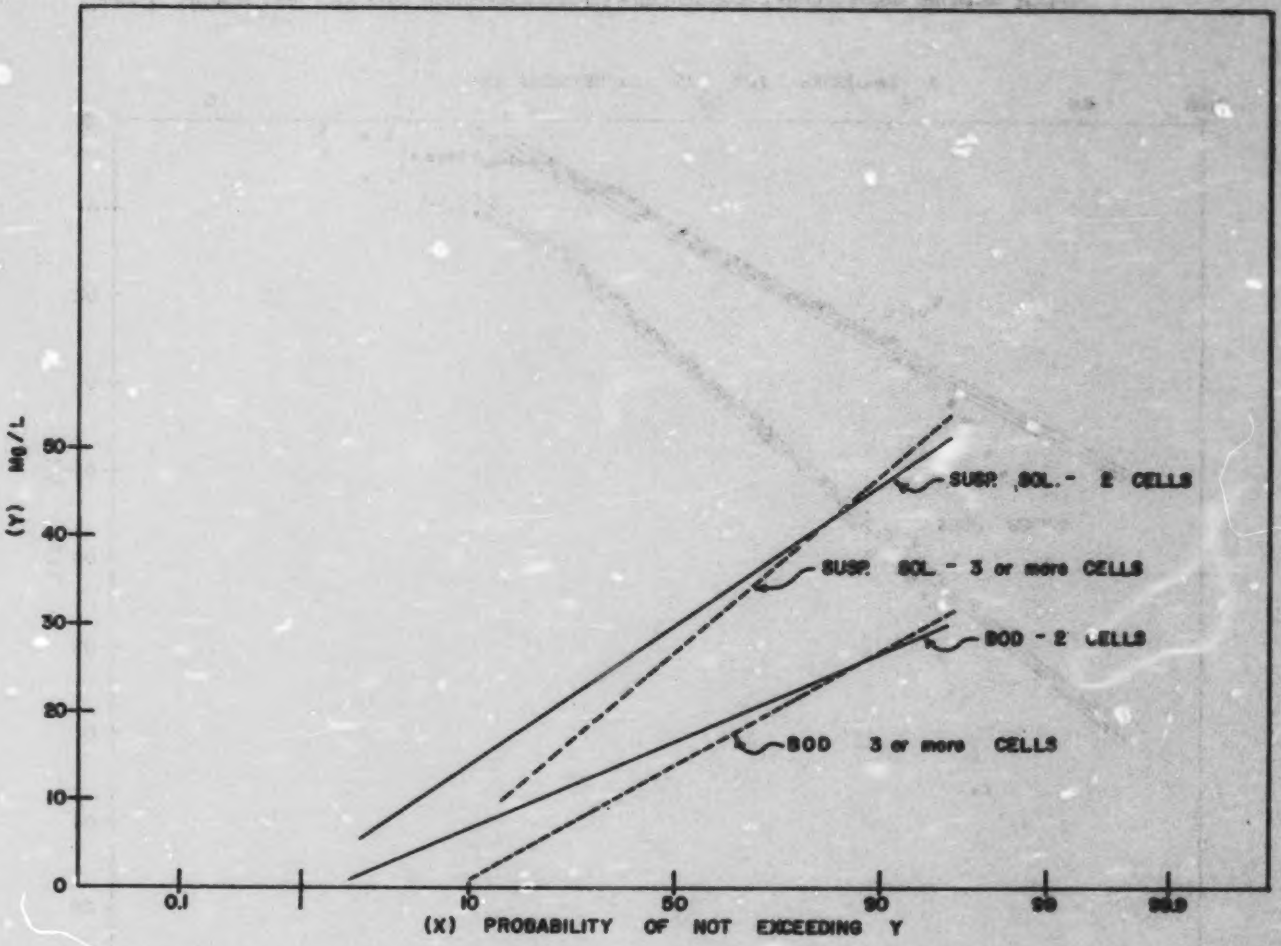

Figure 2. Comparison of effluent quality 2-cell systems vs 3 or more with long period storage before discharge, Michigan.

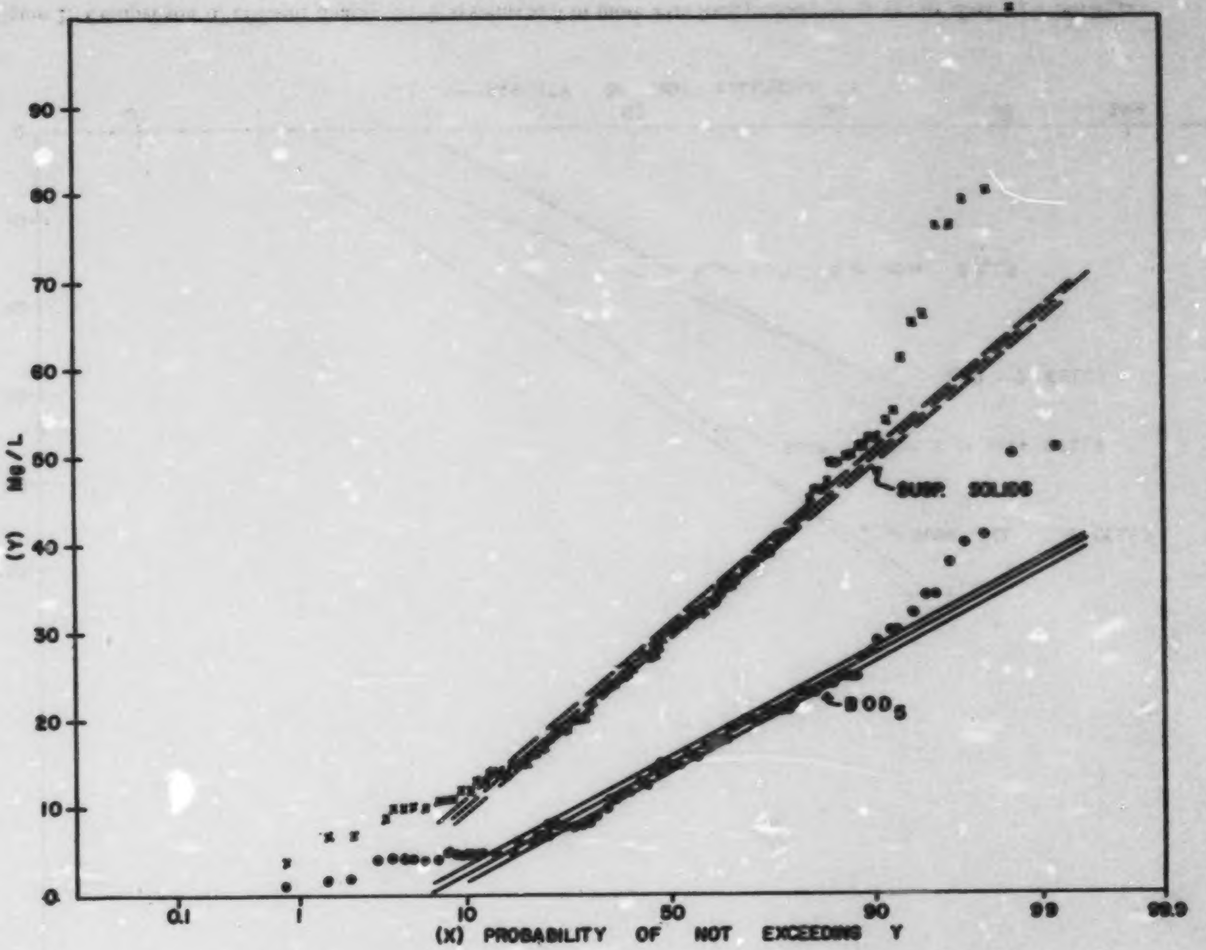

Figure 3. Effluent quality 49 waste stabilization lagoons with long period storage before discharge, Michigan.

SUPPLEMENTARY STUDIES OF LAGOONS OF VARIOUS TYPES

The preceding report sets forth the results of studies, conducted by this investigator during the fall of 1973, of performance of 49 municipal lagoon systems in Michigan treating raw wastewater. These systems were selected for study because, individually and collectively, they typify features of construction, loadings, and maintenance and operational practices which readily accommodate a common mode involving long period storage followed by control short period discharge. Monthly records of one to two years duration, supplemented by chemical and bacteriological data on samples collected during discharge periods, provide an excellent indication of the performance capability of the process and means by which the adverse effects on the process of prolonged periods of ice cover common to many northern states may be overcome.

Supplementary studies were undertaken during the summer of 1974 with the following objectives: To evaluate performance of raw waste stabilization lagoons in other northern states when managed in essentially similar ways; to examine the methods in use and results of chlorination for coliform bacteria destruction both at lagoons so managed and those with continuous discharge; and to collect available information on nitrification phenomena. Results of these studies led to an examination of performance of lagoons used as oxidation ponds following trickling filters at several typical installations and observations of performance of an aerated lagoon followed by chemical precipitation for phosphorus removal and by chlorination for coli destruction.

Supplementary Studies of Raw Waste Stabilization Lagoons with Long Period Storage and Intermittent Discharge

Several of the northern states have adopted a program with criteria for design and operational control of raw waste stabilization lagoons very similar to those in Michigan. Typical of these are the Minnesota and North Dakota programs.

Minnesota

As in Michigan the Minnesota Pollution Control Agency has established criteria for raw waste stabilization lagoon loadings in the 100 person per acre range with provision for long period storage, restricting discharge of effluents to early spring and late fall with total retention from May 15 to October 31 and during winter months. Discharge is authorized by the state agency after testing of samples of lagoon contents at point of discharge show BOD_5 not greater than 25 mg/l, total suspended solids not greater than 30 mg/l, and fecal coli concentrations not to exceed 200 MPN/100 mls. Present requirements call for sampling prior to and during discharge and reporting of results using the standard state report forms.

Some elements of the state program essential to effective performance are the following, excerpted from their *Recommended Procedure for Sampling and Discharging of Wastewater Stabilization Ponds.*

> Prior to any impending pond discharge a representative pond sample(s) shall be taken to determine pond effluent quality. This sample shall be analzyed for 5-day biochemical oxygen demand (BOD_5), Total Suspended Solids (TSS), and fecal coliform bacteria content. In municipalities with unusual industrial wastes other tests by special request of the Agency may be required.
>
> All pond discharges shall have a 5-day BOD_5 concentration not to exceed 25 mg/l, a TSS concentration not to exceed 30 mg/l, and a fecal coliform bacteria count not to exceed 200 most probable number (MPN/100 ml).[2]
>
> Notification of proposed discharge of the ponds shall be made to either the district representative in your area or the Section of Municipal Works (MPCA). The request can be written or by telephone and must include the analytical results of the representative pond sample. Should the results of analysis of the required samples exceed effluent standards, the sampling procedures shall be repeated until effluent standards can be met. Effluent may be discharged only when it is in conformance with the applicable regulations and terms of the permit issued for the disposal system.
>
> After notification, and if there is no objection by the staff, discharge should be accomplished by properly utilizing gate adjustment in the outfall structure. Utilization of gate adjustments should allow the pond to be lowered down to the 18-inch to 1 foot level. At no time should the pond be lowered beyond the 1 foot of depth level.

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[2] If fecal coliform group organisms exceed 200 MPN/100 ml, the effluent should be disinfected by use of chlorine or other approved methods to bring it within standards.

> Caution should be used when discharging so a minimum of bottom scouring occurs. A rapid or uncontrolled discharge could cause bottom scouring, thereby removing the seal.
>
> Calculation of needed storage capacity should be made by the operator. If more storage capacity is required than provided for by one release of a secondary pond, a refilling of the secondary pond and a repeat of procedures should be undertaken (NOTIFICATION FOR SECOND DISCHARGES IS ALSO REQUIRED).
>
> When refilling the secondary from the primary for a second discharge, the operator should only utilize gates in the control structure. Draw off from the primary should be conducted carefully because short circuit effects can occur if the refilling is too rapid or bottom discharge valves are utilized. Improper draw down of the primary could cause poorly treated wastes to fill the secondary pond, thereby lowering the quality in the secondary pond so that further time would be needed to stabilize, thereby causing further delay before tests show the quality is satisfactory for a second discharge.
>
> A *minimum* of two weeks of actual discharging is recommended. This does not include time for testing between discharges. The discharge should be slow and continuous in nature with daily adjustment of the outfall gates.
>
> During discharge testing ... listing shall be undertaken to substantiate effluent quality, i.e., twice a week until the discharge is complete. The results of tests during discharge should be submitted on the next regular monthly report form.
>
> Ponds do not operate themselves; they require proper operation and regular maintenance if extensive and expensive renovations are to be eliminated.

The state agency summarized the results of laboratory analyses of samples collected at typical municipal raw wastewater lagoons during fall 1973 and spring 1974. In the fall, BOD_5 at 36 of the 39 installations sampled was less than 25 mg/l and suspended solids less than 30. Tests for fecal coli were made on samples from 17 installations and for total coli at 14 other. All fecal coli concentrations were 200 or less per 100 mls and 10 of the 14 tested for total coli were 1000 or less with maximum 3800 for the other 4. Effluent quality for the spring discharge as measured by these parameters was generally similar. Tests were made at 49 municipal installations. BOD_5 at 5 lagoons exceeded 25 mg/l with 3 exceeding 30, the high value being 39. Suspended solids values ranged from 7 to 128 with 16 of the 49 exceeding 30 and 10 greater than 40. Only 3 of the 45 tested for fecal coli exceeded 200 per 100 mls.

North Dakota

Design criteria and modes of operation established by the state regulatory agency (Department of Public Health) are similar in most respects to those in effect in Michigan and Minnesota. One significant difference is the firm requirement, adopted early this year, that at least three cells be provided and that the area in the primary cell be approximately one-half the total surface area of all the ponds. Total organic loading on the primary cell may not exceed 30 pounds of BOD_5 per acre per day and the total loading for all ponds may not exceed 20 pounds per acre per day. Further, the total hydraulic loading, including infiltration and inflow, shall be used to determine the volume required to provide a minimum storage capacity of 180 days between the 2 and 5 foot liquid levels. Although provision must be made for series operation "to meet effluent standards and provide for better nutrient reduction," both series and parallel operation is encouraged to provide desirable flexibility for circumstances when one cell must be taken out of use for repair, enlargement or for some other reason.

The standards state that "For winter storage the operating level should be lowered before ice formation and gradually increased to 5 feet by the retention of winter flows. In the spring, the level in the secondary cell can be lowered to any desired depth providing the liquid meets effluent standards and approval to discharge has been obtained."

Discussions with responsible personnel of the state regulatory agency reveal that lagoon systems built and operated in conformity with these principles can consistently meet EPA requirements for BOD_5 and fecal coli and come within reasonable range of the requirement for total suspended solids. The value of regular operational control in accordance with well established principles is recognized.

Upper Peninsula, Michigan

Several small communities in Michigan's Upper Peninsula have constructed raw waste stabilization lagoons during the last 3 years. Five of these systems had a spring discharge this year following several months storage during the rigorous northern winter with long periods of ice and snow cover. These systems are all approximately at design loading of 100 persons per acre. Operational and maintenance practices conform to those outlined in this report for Michigan communities. None of the systems have chlorination.

Table 1. Discharge data, Upper Peninsula Lagoons, Spring 1974.

Community	Date (1974)	BOD_5 (mg/l)	Suspended Solids (mg/l)	Fecal Coli per 100 mls
Bessemer Township	Ice breakup began 4/10; zero ice 4/24			
(2 cells)	5-7	–	–	< 100
	5-16	10	10	< 100
	5-17	16	32	< 100
	5-18	13	13	< 100
	5-19	19	16	< 100
	5-20	18	23	< 100
Bergland Township	Ice breakup began 4/22; zero ice 4/26			
(3 cells)	5-1	No Reliable Data	72	< 100
	5-3		82	< 100
	5-13		60	< 100
	5-16		30	< 100
Wakefield	Ice breakup began 4/19; zero ice 4/23			
(2 cells)	4-24	15	21	100*
	4-29	27	34	2100*
	5-6	21	35	700*
	5-13	25	52	300*
*Total Coli	5-20	23	54	100*
Powers	Ice breakup began 4/8; zero ice 4/18			
(2 cells)	4-25	No Data	No Data	140
	5-7			< 10
	5-9			–
	5-10			–
Gwinn	5-15			< 10
(2 cells)	6-4	No Data	No Data	–
	6-5			–
	6-17			< 10
	6-18			< 10

Belding, Michigan

Studies conducted by the City of Belding on its 5-cell raw waste stabilization lagoons shed additional light on performance of facilities of this type. Of particular interest are the data on progressive nitrification through the several cells operated in series and related levels of BOD_5 and suspended solids. Of interest also are the data on coliform reduction by chlorination of the effluent.

The project is designed for long period storage and seasonal discharge with provision for spray irrigation of the effluent on nearby lands. Studies have been financed by EPA R&D Grant and a wealth of information has been collected for a period of several years.

Facilities

Raw wastes are collected by the municipal sewer system from a population of about 4,000. Some minor industrial wastes are included. At the lagoon site wastes flow through a small anaerobic cell followed by four larger cells operated generally in series. Total surface area is about 55 acres, Cell No. 2 having about 20 acres, Cell No. 3 15 acres, and Cells No. 4 and No. 5 each about 7.5 acres.

Performance

Wastes are generally weak by reason of extensive infiltration of surface water into the sewer system. This is reflected in Table 2 of the Laboratory Analyses Report prepared by the certified resident operator for samples collected on July 21, 1972. These are typical values for late spring, summer, and early fall conditions in the several lagoons. Typical also are the values for samples collected from each of the first four ponds in the series. The quality of the raw waste varies widely depending on rate of infiltration and inflow at the time. Quality as measured by most of the parameters undergoes little change in the fourth and fifth lagoon.

Quality of the effluent from Pond No. 5 during cold weather conditions is indicated in Table 3. Samples were collected during two periods of discharge of the chlorinated effluent to the river. Several trends may be noted. During the January discharge BOD_5 dropped to a very low level. Suspended solids concentrations also were markedly lower, probably by reason of reduced algal concentration. Ammonia nitrogen was very high compared with warm weather levels, continuing to increase as water temperatures became critically low in late January. Phosphorus concentrations increased gradually over the period. No coli were found in the chlorinated effluent.

Characteristics of the effluent from Pond No. 5 discharged to the river are shown again in the following analyses (Table 4) conducted by the operator for the 1974 spring discharge. All ponds were full and overflowing at that time with no ice cover.

Table 4 indicates that BOD_5 concentrations remain low at about the same level as in the preceding November and that suspended solids concentrations are beginning to rise with increase in algae population. Significantly, ammonia nitrogen has dropped prior to April 26 and continues to drop, rapidly reaching mid-summer levels. Lowering in phosphorus levels is also apparent. Total coli concentrations vary for no apparent reason.

Nitrification

The seasonal fluctuation in ammonia concentrations, associated with changes in water temperature are clearly indicated in Table 5 and Figure 1, data for which were derived from the plant operation reports. Ammonia concentration (NH_3-N) is consistently below 0.5 mg/l during July-September rising gradually through October-January with little nitrification during periods of ice cover mid-January through March, remaining at the 10-15 mg/l level until sometime in May in 1972 and late March in 1973. The lagoon surface was ice covered in 1972 until late April and about one month earlier in 1973.

Table 2. Quality of contents of 5 lagoons operated in series at Belding, Michigan.

Analysis	Influent Raw Sewage	Eff. from Pond No. 1	Eff. from Pond No. 2	Eff. from Pond No. 3	Eff. from Pond No. 4	Eff. from Pond No. 5
D.O.	0.0 mg/l	0.0 mg/l	27.8 mg/l	11.2 mg/l	5.0 mg/l	10.8 mg/l
Ammonia Nitrogen	34.6 mg/l	27.3 mg/l	2.7 mg/l	0.4 mg/l	0.6 mg/l	0.5 mg/l
Nitrate Nitrogen	0.16 mg/l	0.2 mg/l	0.44 mg/l	0.5 mg/l	0.19 mg/l	0.08 mg/l
pH	7.1	7.3	8.6	8.6	8.0	8.6
Total Phosphorus	12.5 mg/l	9.9 mg/l	3.0 mg/l	2.5 mg/l	4.4 mg/l	2.9 mg/l
Ortho Phosphorus	10 mg/l	7.8 mg/l	1.6 mg/l	2.0 mg/l	3.4 mg/l	2.3 mg/l
Suspended Solids	121 mg/l	76 mg/l	146 mg/l	31 mg/l	17 mg/l	22 mg/l

Table 3. Effluent quality–Pond No. 5, late fall and winter discharge.

Date	D.O.	BOD_5	Suspended Solids	NH_3-N	NO_3-N	Total P	Total Coli (MF)
1973							
11-5	10.5	3.0	52	2.4	0.35	2.7	0
11-7	10.7	8.9	60	5.58	0.33	3.9	0
11-13	10.8	10.3	102	5.58	0.41	3.9	0
11-20	9.7	9.4	78	5.82	1.1	3.9	0
11-22	10.0	8.7	52	5.22	0.97	3.5	0
Total Discharge - 57,559,000 gallons							
1974							
1-15	9.7	7.2	9.5	5.7	0.82	3.4	0
1-18	10.9	1.4	11	5.96	0.66	3.6	0
1-22	8.2	5.4	13.5	7.4	0.22	4.0	0
1-25	5.0	1.2	12.5	9.0	0.16	4.4	0
1-29	10.5	2.4	30	10.8	0.15	5.1	0
Total Discharge - approximately 38 million gallons.							

Table 4. Characteristics of the effluent from Pond No. 5.

Date	D.O.	BOD_5	Suspended Solids	NH_3-N	NO_3-N	Total P	Total Coli (MF)
1974							
4-26	17.5	6.7	53	3.3	1.1	2.[illegible]	–
4-29	12.0	10.5	30	3.5	1.0	2.9	<100
5-1	10.7	7.8	16	2.6	1.3	3.0	<100
5-6	9.4	8.9	23	1.0	1.4	3.2	1800
5-7	10.0	9.8	12	0.75	1.5	2.8	–
5-8	10.3	7.0	16	0.8	1.4	2.7	5500
5-13	9.6	9.1	27	0.1	1.0	2.5	<100

Note: All chemical results expressed as mg/l.

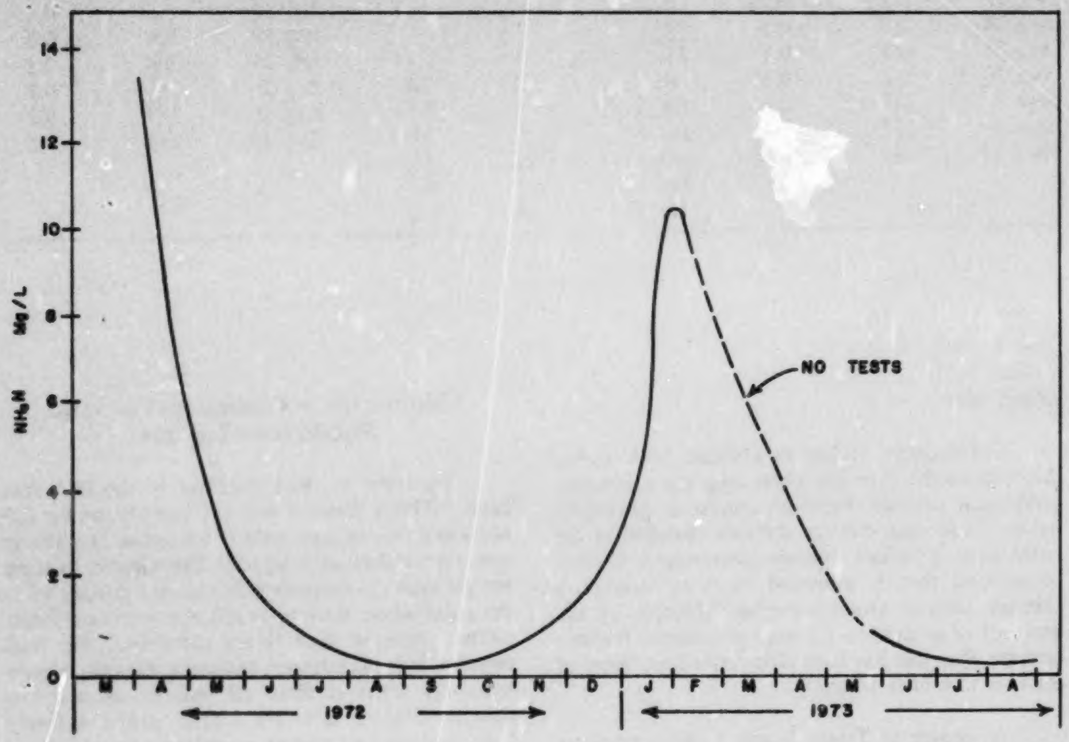

Figure 1. Seasonal variation in ammonia nitrogen lagoon effluent, Belding, Michigan, April 1972 - August 1973.

Table 5. Data showing seasonal variation in ammonia nitrogen in lagoon effluent, Belding, Michigan, 1972-1973.

Date	Day of Year	NH_3 (mg/l)	Date	Day of Year	NH_3 (mg/l)	Date	Day of Year	NH_3 (mg/l)
1972								
Mar. 20	79		Sept. 19	262	0.3	Jan. 22	22	7.4
Mar. 25	84	17.2	Sept. 25	267	0.3	Jan. 25	25	9.0
Mar. 30	89	27	Sept. 29	271	0.3	Jan. 29	29	10.8
Apr. 4	94	19	Oct. 9	282	0.8	Feb. 1	32	11.1
Apr. 14	104	17	Oct. 14	287	1.8	Feb. 5	36	8.5
Apr. 19	109	13	Oct. 19	292	1.7	Feb. 8	39	10.1
Apr. 29	119	13.5	Oct. 25	298	3.5	Feb. 12	43	10.6
May 4	124	8.0	Oct. 30	303	3.6	Feb. 15	46	10.1
May 9	129	6.8	Nov. 15	319	1.5	Feb. 19	50	9.8
June 16	167	2.0	Nov. 18	322		Feb. 22	53	9.6
June 21	172	1.1	Nov. 27	331	0.2	Feb. 26	57	11.1
June 26	177	1.1	Nov. 30	334	0.4	Feb. 28	59	10.4
June 30	181	1.1	Dec. 7	341	1.6	Mar. 16	75	9.6
July 6	187	0.6	Dec. 11	345	0.9	May 24	144	0.4
July 11	192	0.4	Dec. 14	348	1.6	May 31	151	0.5
July 17	198	0.3	Dec. 19	353	1.2	June 7	158	0.1
July 21	202	0.5	Dec. 21	355	2.1	June 14	165	0.4
July 26	207	0.3	Dec. 26	360	2.1	June 21	172	0.3
July 31	212	0.03	Dec. 29	363	2.3	June 28	179	0.3
Aug. 5	217	0.05	Dec. 31	365		July 5	186	0.3
Aug. 15	227	0.04				July 12	193	0.3
Aug. 21	233	0.3	1973			July 19	200	0.2
Aug. 25	237	0.3	Jan. 1	1		July 25	206	0.2
Aug. 30	242	0.3	Jan. 2	2	2.4	Aug. 2	214	0.2
Sept. 4	247	0.03	Jan. 5	5	2.8	Aug. 16	228	0.2
Sept. 9	252	0.2	Jan. 8	8	3.0	Aug. 23	235	0.2
Sept. 14	257	0.06	Jan. 11	11	3.1			
			Jan. 15	15	5.7			
			Jan. 18	18	6.0			

Disinfection

Chlorination of lagoon effluent both during discharge to the river and when irrigating is a regular established practice. Facilities consist of gas chlorinators discharging through diffusers installed at the entrance to a baffled chlorine contact tank. Control of chlorine feed is exercised so as to maintain a chlorine residual (total combined chlorine by OT method) of as close to 0.5 mg/l as possible. Records indicate that this has been done effectively most of the time with little difficulty.

As shown in Tables 3 and 4, these practices usually have been effective in destruction of total coli. Other data at the on-site laboratory indicate that there are seldom any fecal coli found in these chlorinated wastes.

Chlorination at Continuous Discharge Stabilization Lagoons

The July 1, 1974, printout of the EPA data bank (STPOM forms 5 and 12) identify nearly 100 municipal installations with chlorination facilities at raw waste stabilization lagoons. The majority of these use gas type chlorinators with chlorine introduced at the point where the lagoon effluent enters a chlorine contact tank as it is being discharged for final disposal. It is discouraging and quite revealing to note that only seven of these are reported as recording chlorine residuals after the contact period and only four perform bacteriological analysis, two for fecal coli and two for total coli. It is expected, of course, that most if not all of these municipalities will soon be performing such tests to meet EPA regulations and NPDES requirements.

This investigator made a search of the technical literature and talked with program directors in many states, attempting to locate and obtain useful, reliable information on lagoon effluent disinfection practice. The paucity of available data of this kind confirms that only a few municipalities in the nation perform sufficient laboratory tests of this nature to provide a basis either for control of the process or a reliable indication of what is actually being achieved in bacterial control.

In many of the states there are few if any installations depending on effluent chlorination for bacterial control. Rather the natural process of die-off during long period storage under conditions prevailing in the lagoon are depended upon for this purpose. It is clear, however, that under many circumstances of high loading, short residence time and unfavorable climatic conditions, high concentrations of fecal coli have been found and can be expected to be present in the lagoon discharge.

It is of interest, therefore, to examine the data available on effluent disinfection practice and results.

As indicated herein the City of Belding, Michigan, has established an effective method of disinfecting, with chlorine, effluents discharged from a system of five lagoons operated in series after long period storage. No data are available at this location for short period storage or continuous discharge.

Chesterfield Township studies

The City of Detroit operates a raw waste stabilization lagoon facility serving limited residential areas in Chesterfield Township, Macomb County. This facility is designed and operated for continuous discharge with effluent disinfection. There are three lagoons each of about equal size with a total area of about 70 acres. Water depth is maintained constantly at about 6 feet.

Raw wastewater is delivered to the lagoon site by force main from a remote site. Daily flows fluctuate between 0.5 mgd and 2.0 mgd, influenced by infiltration and inflow to the sewer system. The wastes are directed through the three cells in series, then flow by gravity through a chlorine contact tank after initial mixing with chlorine from two 1-ton cylinders. Chlorine is usually fed at a constant 90 lb per day rate. Chlorine residuals after 5 minutes contact are always at least 1.0 mg/l and usually higher.

Alum is fed into the pipe connecting Cells 2 and 3 at a constant rate of 6½ gallons per hour (55 mg/l Al_2SO_4). This is found to assist materially in clarification of the contents of the final cell.

Samples of the effluent are collected twice daily for chemical and bacterial analyses and chlorine residuals are determined on each sample. The data reported by the City of Detroit wastewater treatment plant operations staff are summarized in Table 6.

Table 6. Effluent quality 3-cell continuous flow lagoons, Detroit (Chesterfield Twp.), Michigan, January 1973 - May 1974.

Month	pH	BOD_5 (mg/l)		Susp. Solids (mg/l)		Total P	Fecal Coli
		Range	Avg.	Range	Avg.	mg/l	per 100 mls
1973							
Jan.	6.5-6.8	1-9	4	11-82	34	2.8-6.0	<23-230
Feb.	6.7	3-13	9	17-124	35	5.2-8.4	<23
Mar.	6.2-8.4	5-14	10	10-200	68	5.5-7.2	<23
Apr.	7.7-8.7	2-7	4	18-126	47	3.4-6.1	<23
May	8.1-9.0	2-24	10	8-50	31	3.0-6.9	<23
June	7.8-8.6	4-26	16	14-102	61	3.2-4.8	<23
July	7.0-8.3	2-16	11	19-49	47	1.5-4.1	<23
Aug.	7.4-8.5	1-21	5	5-68	31	1.6-3.0	<23
Sept.	7.9-8.6	0-19	4	3-83	34	0.5-2.5	<23
Oct.	7.3-7.9	0-16	4	2-38	15	2.2-6.3	<23
Nov.	7.6-7.9	0-11	2	4-29	11	1.6-2.9	<23
Dec.	7.8-8.2	1-4	2	3-47	14	2.7-4.5	<23
1974							
Jan.	7.6-8.1	1-8	2	4-19	7	4.2-6.7	<23-11000
Feb.	6.8-7.5	5-36	19	9-82	39	1.9-6.3	23-2300
Mar.	7.7-8.5	3-29	9	8-36	21	3.0-7.6	<23-730
Apr.	7.2-8.1	1-4	2	15-66	35	1.7-3.1	<23
May	7.5-8.7	1-13	1	13-58	41	1.6-2.8	<23

No reliable data are available on fecal coli concentrations of lagoon effluents prior to disinfection although spot samples taken from time to time indicate that concentration is variable, ranging from less than 10 to over 1000 per 100 mls.

The effluent samples are routinely tested for total phosphorus. Concentrations range generally from 1.5 to 6.0 mg/l as P. No data are available on phosphorus levels in the raw waste.

Illinois studies

The Illinois Environmental Protection Agency assembled information for this report from their data printout system on nine municipal waste stabilization lagoons with chlorinated effluents. All of these systems are of the continuous flow-through type. Present loadings are 90-105 persons per acre (total acreage) except for facility No. 4 which had about 160 persons per acre when sampled. This facility has relatively small volume and surface area in Cells 2 and 3.

Samples are collected and analyzed by the agency. Results are summarized in Table 7.

No information is available at this writing on the type of chlorination facilities, chlorine feeding, and mixing and contact tanks, nor is there information on chlorine feed control related to chlorine residual levels and coliform concentrations. Such data are needed to determine the capability of the facilities to achieve low fecal coli levels dependably.

The BOD_5, suspended solids concentrations are generally a bit higher than noted for the long period storage and controlled discharge lagoons reported here but ammonia and nitrate levels are very similar to these. Here again high ammonia levels were associated with cold weather and lowest levels with summer temperatures.

Chlorination of Lagoon Contents

A small residential subdivision known as Red Oaks of Chemung is served by a wastewater treatment system consisting of 2-cell lagoons followed by spray irrigation. The facilities are operated by the County Department of Public Works.

The two cells are operated in series. All raw wastewater is delivered to Cell No. 1 and transferred by pumping to Cell No. 2 where the contents are isolated for several weeks before chlorination of the total liquid surface area prior to release for spray irrigation.

The following procedure, established by the consulting engineer who designed the facilities, is utilized by operating personnel to assure effective disinfection.

> Irrigation from the north lagoon shall not be carried out until the following chlorination procedures have been accomplished. Twenty-four hours prior to irrigation an application of at least 8 mg/l of available chlorine shall be applied to the lagoon. The application shall be as evenly distributed as possible by broadcasting. Two hours before irrigation a second application of at least 2 mg/l of available chlorine shall be similarly accomplished. Just prior to each chlorine application at least two samples shall be collected and a dissolved oxygen test accomplished with the results being recorded on the data forms. Before pumping to irrigation a sample shall be collected and analyzed for a free chlorine residual—no spraying shall be accomplished unless a substantial chlorine residual is displayed and duly recorded. These chlorine tests shall be conducted at least twice during the period of pumping to determine that a chlorine residual is being maintained. If no residual can be detected, spraying shall be halted.

When a chlorine residual has been established and maintained, with additional spray-

Table 7. Summary by Illinois EPA analysis of results from nine municipal waste stabilization lagoons with chlorinated effluents.

Facility No.	No. Samples	BOD_5 (mg/l) Range	Avg.	Susp.Sol.(mg/l) Range	Avg.	NH_3-N (mg/l) Range	Avg.	NO_3-N (mg/l) Range	Avg.	Fecal Col. (MPN) Range	Geom. Mean
1	7	1-37	8	4-74	26	0.7-7.0	2.1	0-0.3	0.1	100-600	184
2	8	4-27	19	6-80	45	0.2-1.2	0.5	0-0.8	0.2	100-1700	237
3	2	13-27	21	11-35	26	0.4-1.8	1.1	0.1-0.7	0.4	0-100	10
4	6	18-50	35	70-160	105	0.3-3.1	1.0	0-0.5	0.2	100-3400	546
5	9	1-38	9	1-19	8	0.1-5.9	1.0	0-0.6	0.1	10-30000	272
6	7	15-45	25	13-88	60	0.2-8.3	2.7	0-1.0	0.3	10-1000	116
7	3	0-25	12	1-58	25	0.2-7.1	2.5	0.2-0.3	0.2	50-100	62
8	9	12-46	23	3-66	38	0.2-3.0	0.9	–	–	0-1300	19
9	3	9-33	21	22-92	54	0.3-2.5	1.0	0-1.9	0.6	0-140	5

ing contemplated for the following day, the 24 hour prior application of 8 mg/l will not be essential. However, the two hour prior 2 mg/l application shall be accomplished.

Chlorine applications shall be in accord with the water level in the lagoon and the computed quantities tabulated below.

		Pounds Chlorine Required			Pounds Chlorine Required		
Liquid Depth in Feet	Volume in gals. x 1,000	2 mg/l Application	40% Powder	70% Powder	8 mg/l Application	40% Powder	70% Powder
1	255	4	10	6	16	41	23
2	530	9	21	12	34	85	48
3	825	13	33	19	53	132	75
4	1,143	18	46	26	73	183	105
5	1,485	24	60	34	95	238	136
6	1,852	30	74	42	119	296	169
7	2,244	36	90	51	144	359	205
8	2,662	42	107	61	170	426	243
9	3,107	50	124	71	199	497	284

The hypochlorite powder is distributed manually with a rotary-type fertilizer spreader mounted in a row boat. One man cranks the spreader while another rows the boat back and forth until the entire surface has been well covered.

Samples for bacterial analyses are collected by the public works staff and sent to the state laboratories by sample mailers. Fifty-seven samples were analyzed for total and fecal coli, 31 of which were collected mornings, and 26 during afternoons on 31 days during irrigation periods. Total coli concentrations ranged widely from a few thousand to a few hundred thousand with a geometric mean of 8984. On the other hand fecal coli levels were less than 10 per 100 mls for 50 of the 57 samples with a geometric mean of about 10.

This appears to be a most effective method of disinfection for small installations.

Lagoons Following Secondary Treatment

One hundred and ten lagoons following trickling filters and activated sludge plants are recorded in the July 1 EPA data bank printout of facilities in the federal grant program. It is to be expected that many more such facilities will be constructed to meet present and future effluent requirements. It is useful, therefore, to examine the great wealth of information generated in the study of performance of lagoons following trickling filters at the State Prison of Southern Michigan.

State Prison of Southern Michigan

The treatment works serving the prison were built in the late 1930's and have continued in service without major modifications until polishing lagoons were installed in 1968 to meet more stringent requirements for discharge to a critically oxygen-deficient stream receiving wastes from this and other sources including the City of Jackson.

Facilities

Raw wastes enter old Imhoff Tanks after screening and grit removal, thence to trickling filters from which the effluent is discharged to a 2-cell lagoon normally operated in series. A yearly management pattern of storage in and discharge from the lagoons is utilized for controlled release to the Grand River when flows and dissolved oxygen in the river are relatively high and lagoon quality is optimum. In a typical year storage without discharge begins in May as river flow decreases and continues until about November when lagoon contents are lowered in series from about 10-foot depth to about 3 feet. When water levels reach 10 feet again in December, the lagoons are discharged in series under ice cover until some time in April after ice has thawed when water levels are again lowered to about 3 feet and made ready for summer storage.

At the 10-foot operating level with cells in series residence time at 1.1 mgd flow is about 120 days.

No aeration is provided. Wastes are not chlorinated at any point in the process.

Performance

The following data (Table 8) are taken from operation reports submitted each month to the state regulatory agency by the certified plant superintendent. Observations and laboratory testing of a very thorough and varied nature are made daily throughout the year by the operating staff.

During summer months ammonia nitrogen concentrations are typically around 0.5 mg/l as N, rising to about 2.0 with coldest water temperatures.

Analyses are made routinely for total coli in the lagoon effluent or near the point of discharge. Generally concentrations are very low. A large percentage of tests were registered as zero and seldom exceeded 200 per 100 mls. During four days in September and five in October the report indicates TNTC (too numerous to count).

Table 8. Performance data of lagoons following trickling filters at State Prison of Southern Michigan at Jackson.

Item	Period December 1971		Period Year 1971	
	Daily Range	Mo. Avg.	Monthly Range	Ann. Avg.
Raw Flow (mgd)	0.9-1.2	1.06	1.04-1.22	1.11
BOD_5 (mg/l)				
Raw	145-340	228	156-237	200
T.F. Eff.	13-34	22	13-22	17
Lagoon Eff.	4-11	8	4-22	11
Susp. Sol. (mg/l)				
Raw	160-248	183	164-184	176
T.F. Eff.	24-50	34	34-48	38
Lagoon Eff.	16-42	29	18-93	43
pH				
Raw	7.1-7.4	7.2	7.0-7.2	7.1
Lagoon Eff.		8.4	7.9-9.8	8.8
Total Phosphorus (mg/l as P)				
Raw	4.5-8.7	6.5	4.8-7.2	5.8
T.F. Eff.	4.3-6.0	5.4	3.6-5.7	4.6
Lagoon	0.8-2.3	1.8	0.1-2.5	1.1
NH_3-N (mg/l)				
Raw	10-19	13.6	9.1-13.7	10.9
T. F. Eff.	3.5-7.0	5.3	2.8-5.3	4.1
Lagoon Eff.	0.6-2.1	1.1	0.2-1.8	0.8

Fargo, North Dakota

Another example of lagoons installed to supplement treatment provided by trickling filters are those completed by the City of Fargo a year or so ago to meet elevated effluent requirements for discharge to the Red River of the North.

Facilities

Raw wastewater averaging 5-6 mgd, collected by the municipal sewer system, passes through a screening chamber, primary settling tanks, and two trickling filters, one with stationary nozzles, the other with rotary distributors, followed by final settling tanks. Sludge digestion facilities complete the mechanical plant portion now in use. Chlorination facilities customarily used in the past have not been used since the lagoons were placed in operation last fall. The wastes from the plant are pumped some 4 miles to the lagoon site which occupies one square mile. Lagoons consist of five cells, four of which are used as primary and secondary units on a selective basis and the fifth, deeper than the others is used as the final polishing unit prior to discharge.

Performance

The final cell normally is filled and isolated with no inflow for several weeks before the contents are released. Storage takes place through the long winter months without discharge. When the quality of the contents of Cell No. 5 are favorable for discharge in the spring, the cells are progressively lowered. This process continues through summer and fall, drawing all cells down to lowest level consistent with good effluent quality before the onset of winter weather.

Effluent quality is typically as follows:

Trickling filter plant
BOD_5 30 - 40 mg/l - mid-spring to mid-fall
60 - 70 mg/l - cold weather
Suspended solids - A little higher than BOD
Lagoons
BOD_5 10 - 15 mg/l, sometimes less
Suspended solids - 10 - 20 mg/l usually
Fecal coli < 100 per 100 mls consistently

Aerated Lagoons

The EPA data bank on facilities constructed or proposed has record of 137 aerated lagoons as of July 1, 1974. These vary widely in construction features and loadings and are designed to achieve a rather wide range of effluent quality, either by themselves or in combination with other treatment units.

Reed City, Michigan

A quite typical installation of this kind has been in operation at Reed City, Michigan, since late 1971. These facilities were selected for study because they typify the rather common circumstance of a small community with a large but fluctuating organic loading from industry where the treated wastes discharge to a quite limited stream resource, requiring a high quality effluent low in oxygen consuming substances, nutrients, and coliforms. Industrial wastes consist of 30-40,000 gals/day of rinse water following withdrawal of strong whey in cottage cheese manufacturing plant. BOD_5 of these wastes ranges from 3000-4000 mg/l. The mixed wastes have BOD_5 averaging about 350 mg/l and total suspended solids averaging about 160 mg/l.

Facilities

Raw wastes are discharged to three 1-acre aerated lagoons approximately 10 feet deep (operating depth) equipped with 45,000 feet of perforated piping (Hinde System). Lagoons are operated in series. Residence time in each cell at present loadings is about 10 days.

The lagoon effluent is returned to the old plant site for phosphorus removal and disinfection. Chemicals (lime or alum) are added in a rapid mix chamber, thence to a flocculation chamber where a polymer is added prior to discharge to the old sedimentation tanks. Chlorine is applied by semi-automatic gas chlorinators to the settled effluent prior to entering the old baffled chlorine contact tank 10-25 minutes time of passage at present flow rates. 3000-4000 gallons of chemical sludges are withdrawn daily from the sedimentation tanks to a small sealed earthen lagoon and periodically removed for land disposal.

Performance

The data in Table 9, extracted from the October 1972 and June 1973 monthly operation reports submitted by the certified operator to the state regulatory agency, are typical of loadings, chemical dosages and effluent quality during 1972 and the present time.

It is of interest to compare these results with performance of the original primary plant which consisted of grit removal, screening, plain sedimentation tanks, chlorination and heated sludge digesters. The operation report for September 1971 provides data for these facilities.

Item	Range	Mean
Flow (mgd)	0.23-0.31	0.28
BOD_5		
Raw	250-1120	557
Eff.	150-400	304
Suspended Solids		
Raw	150-322	215
Eff.	72-180	134
Chlorination		
Cl_2 feed (lbs/day)	88-123	105
Residual (mg/l)	16-30	
Coli, total per 100 mls	0-1500	

Table 9. Performance data of aerated lagoons followed by chemical precipitation, sedimentation, and chlorination at Reed City, Michigan.

Item	October 1972		June 1973	
	Range	Mean	Range	Mean
Flow (mgd)				
Raw	0.20-0.30	0.24	0.28-0.37	0.33
Eff.	0.18-0.34	0.26	0.28-0.47	0.35
BOD_5 (mg/l)				
Raw	160-600	408	100-460	306
Eff.	1-6	3	1-13	8
Suspended Solids (mg/l)				
Raw	126-206	163	110-186	159
Eff.	6-36	18	7-25	17
Total Phosphorus (P) in mg/l				
Raw	10-21	18	6-17	13
Lagoon Eff.	13.5-15.5	14.3	12-13.5	13
Eff.	1.3-5.6	2.4	2.6-12.2	6.7
Chlorination				
Cl_2 feed (lbs/day)	5-9	6.7	8-17	13
Residual (mg/l)	0.4-1.0		0.8-1.5	
Coli, total (per 100 mls)	0-7.3		4-43	

Operational control

The treatment facilities and laboratory analyses are under the control of the same certified operator who capably operated the primary plant before, during and after construction of the modifications and additions. He has one assistant operator. Between them they manage the facilities 7 days a week. Some indication of the continuity of control is the recorded data for chlorination. At least 6 readings, usually 8, between 7 a.m. and 5 p.m. are taken daily on rate of treated waste flow and chlorine feed rate and chlorine residuals are determined for each reading. Chemical analyses are made, usually 4 days per week, and many physical measurements and observations are recorded daily. Operators perform all maintenance and are intimately familiar with all equipment and process control measures. When questioned about attendance on Saturday and Sunday the certified operator said, "It's no different than managing a dairy herd. You've got to milk them twice everyday."

Observations and Conclusions

1. *Long period storage and controlled discharge.* Study of the history of performance of raw waste stabilization lagoons with an effective discharge control program involving long period storage and seasonal discharge, including over 40 installations in Minnesota and many in North Dakota, confirm the observation made last year on the study of 49 Michigan installations of this kind that: (1) Such lagoons can consistently produce effluents meeting EPA requirements for secondary treatment for BOD_5 and fecal coli and generally can meet the suspended solids requirement when loadings are in the 100 persons per acre range, and (2) fecal coli requirements of 200 and 400 per 100 mls cannot be met with such facilities during winter ice cover and when water temperatures approach freezing temperatures. Again no data were available on performance of these facilities at loadings above 20-25 pounds of BOD_5 per acre per day (based on total acreage) or short retention periods.

2. *Continuous flow systems.* Nine continuous flow lagoons in Illinois with system loadings in the 100 person per acre range generally met the EPA secondary treatment requirements for BOD and suspended solids.

3. *Nitrification.* Studies of a 5-cell lagoon system at Belding demonstrated a high degree of nitrification in the first three cells with ammonia nitrogen falling from some 20-30 mg/l in the raw waste to less than 1 mg/l in mid-summer but with practically no nitrification during winter months. Ammonia levels in the effluent fell in late spring and rose in early fall in a cyclic fashion. Similar results were found at nine typical continuous discharge lagoons in Illinois which were selected for study by reason of their data on effluent disinfection. This temperature relationship was found in lesser degree at the State Prison of Southern Michigan at Jackson where a 2-cell lagoon system with controlled discharge is used to improve a trickling filter effluent. Here ammonia concentrations of around 0.5 mg/l as N rise to about 2.0 mg/l during winter discharge.

4. *Chlorination for disinfection.* Little information could be found on fecal coli concentrations following disinfection of the lagoon effluent with chlorine although over 100 such installations are recorded in EPA's O&M file of facilities inspected. Data from the nine continuous flow lagoons in Illinois indicated fecal coli levels approaching the required 200 per 100 ml standard but with a fair percentage considerably higher. More study of facilities of this kind is needed to determine what can be achieved in such installations. Chlorination of the effluent from a 3-cell continuous flow lagoon operated by the City of Detroit with chlorine residual levels consistently above 1 mg/l produced a high degree of kill. Fecal coli concentrations were nearly always less than 23 per 100 mls. Excellent results were obtained also from the 5-cell controlled discharge lagoon at Belding, Michigan, with chlorine residual concentrations consistently around 0.5 mg/l after a 10-25 minute contact period.

5. *Lagoons following trickling filters.* Several years of good records at the Southern Prison of Michigan with two-stage lagoons with up to 4 months residence time following standard rate trickling filters document effluents have BOD_5 usually less than 10 mg/l, total phosphorus under 2 mg/l most of the year, a high degree of nitrification during warm weather and very low fecal coli concentrations. Results at a similar quite new installation at Fargo, North Dakota, indicate promise of similar performance.

6. *Aerated lagoons.* An aerated lagoon followed by chemical flocculation for phosphorus removal at Reed City, Michigan, routinely produces an effluent with BOD_5 less than 10 mg/l and suspended solids under 20 mg/l. Fecal coli in the effluent are consistently under 50 per 100 mls with low chlorine feed rates and chlorine residuals of 0.5 to 1.5 mg/l.

7. *Number of cells required.* In our study of performance of 49 raw waste stabilization lagoons last fall it was observed that little difference could be detected in the quality of effluent from 2-cell compared with 3-cell systems. It was noted also that all of the systems studied were either at or below the design loading of 20 pounds BOD_5 per acre per day or 100 persons per acre. Generally, these facilities receive more attention and are more effectively

operated by specially trained public works personnel than usually found elsewhere. Discharge occurs only after long period storage and usually after a period of several days or weeks of isolation of the discharging cell. It was further noted that nearly all of the 49 systems were relatively young, having been in operation for 2 to 5 years and some even less.

Each of these factors provide some increase in the chance that the effluent from a second cell be of good quality. It is reasonable to expect, however, that over a longer period of time the greater flexibility afforded by a third cell will increase the capability of the system to operate continuously at a high level of effectiveness. The value of the third cell will be greatly increased with provision for parallel or series operation so that the wastes may be routed into or out of any of the three cells. Experience throughout the country clearly indicates that any one of the cells may require isolation, drawdown for repairs or special maintenance, often requiring long periods to correct the problem. For these reasons many of the states have adopted standards or guidelines calling for a minimum of three cells for raw waste stabilization lagoons, a decision well founded on practical considerations.

8. These studies indicate that various types of lagoon facilities with conservative loadings and effective operational control are producing effluents which meet secondary treatment requirements and hold promise of a high degree of nitrification and other evidences of a high quality effluent. It is to be expected that improved operational controls including regular laboratory analyses will greatly improve the performance of a high percentage of lagoons now in use, at the same time providing a firmer base for design of facilities for the future.

SEPARATION OF ALGAE CELLS FROM WASTEWATER LAGOON EFFLUENTS BY SOIL MANTLE TREATMENT

R. A. Gearheart and E. J. Middlebrooks[1]

Soil Mantle Disposal of Lagoon Effluent

A review of the history of sewage treatment indicates that wastewater irrigation was originally developed in the early nineteenth century as a system of both treatment and disposal (Rafter, 1897; Rudolfs and Cleary, 1933; Mitchell, 1931). In recent years, other forms of waste treatment have become popular. The growth of technical knowledge has led to a reexamination of the possibilities of treatment and disposal of certain industrial, agricultural, and domestic wastewaters through the application of irrigation techniques (Riney, 1928; Mitchell, 1930; Goudey, 1931; McQueen, 1934; DeTurk, 1935). Thus the cycle has run its course, the earliest form of waste treatment is now the subject of research and development for the disposal of the water of our time (Skulte, 1956; Dye, 1958; Henry et al., 1954; Hunt, 1954; Warrington, 1952; Wells, 1961).

However, irrigation with wastewater is not a panacea for the economical treatment and disposal of wastes. Sanitary, aesthetic, economic, ecological, and other technical and practical considerations must be carefully balanced for a sound wastewater irrigation system.

The effect of sewage effluent on the yield of agronomic crops has, in most cases, been found to be beneficial (Hill et al., 1964; Herzik, 1956; Merz, 1956; Wilcox, 1949). Feinmesser (1970) obtained a significant increase in the yield of reed canary grass. Huekelekean (1957) obtained excellent crop yields in Israel. Stokes et al. (1930) obtained yield increases in Florida amounting to 240 percent for both Napier grass and Japanese cane, when compared with the unirrigated crops, or with the same crops irrigated with well water. Day and Tucker (1960a, 1960b) and Day et al. (1962) in Arizona, obtained beneficial yield effects on small grains which were harvested as pasture forage, as hay or as grain. Parizek et al. (1967) at Penn State, have extensively explored the use of wastewater as a spray irrigant on forestry land.

More than 100 kinds of viruses are known to be excreted by man and approximately 70 of these have been found in sewage (Clark and Chang, 1962). Those that appear to be transmitted through wastewaters are the entero-viruses, poliomyelitis (Paul and Trask, 1942a, 1942b; Little, 1954; Kelley, 1957; Bancroft et al., 1957), coxsachie (Clark, 1971) and infectious hepatitis (Hayward, 1946; Dennis, 1959; Yogt, 1961). There are a limited number of studies of the movement of viruses through granular media (Merrell, 1963). These studies showed that rapid sand filtration preceded by coagulation and sedimentation only partially remove virus. The removal of virus from percolating water is largely due to sorption on the soil particles. Soils having a higher clay content sorbed virus more rapidly than those with less clay (Eliassen et al., 1967).

The soil system is composed of gas, water microorganisms, minerals, and organic matter which form the solid matrix. Experience has indicated that this dynamic system is constantly undergoing physical, chemical, and biochemical interactions. Wastewater applied to the soil mixes with the existing soil water, becoming a part of the system, and may alter the nature and rate of change of the physical, chemical, and biochemical processes in the soil system.

Many insoluble constituents such as suspended minerals, particulate organics, and inorganic precipitates are quickly removed from the liquid by the surface area of the soil. Some of these substances are altered, but some become a permanent part of the soil. Irrigation with wastewaters may produce beneficial or detrimental changes in the soil system.

Physical clogging of the soil pores and the resulting loss in the infiltration rate have caused many wastewater soil treatment systems to fail (Avnimelech and Nevo, 1964; Jones and Taylor, 1965; Mitchell and Nevo, 1964; Winneberger et al., 1960; Thomas et al., 1966; Amramy, 1961). The political hazard of high sodium rates to the physical properties of certain soils is of paramount concern. This phenomenon has

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been studied extensively for the improvement of saline and alkali soils by the proper management of irrigation practices (U.S. Salinity Laboratory, 1954). It is well known that additions of organic matter improve the aggregate stability of soils, and wastewaters high in organics have been used to improve the physical properties of soils (Merz, 1959).

It has been known for years that organic matter serves as a granulating agent in soils. Bauer (1959) showed that organic matter is conducive to the formation of relatively large stable aggregates and that the effect of organic matter is more pronounced in soils containing small amounts of clay. Small amounts of added organic matter appear to promote large stable aggretages of clay, silt, and sand. The organic content of oxidation pond effluent, both dissolved and particulate (algae) may therefore have a beneficial effect on the soil permeability. Soil microorganisms undoubtedly play a major role in producing organic cementing materials. Martin and Waksman (1940) observed that the growth of microorganisms led to the binding of soil particles, and the more readily the substrate on which the microorganisms bred decomposed, the greater the effect on aggregation. Plant roots appear to be very effective in promoting aggregation by soils. The unusual aggregation of soils in the roots of plants probably is the consequence of mechanical disturbance by plant roots and by wetting and drying, together with cementation by organic compounds (Jenny and Grossenbecker, 1963). The efficiency of spray irrigation of vegetated areas for wastewater disposal is undoubtedly due in part to enhancement of permeable structures by plant roots.

Filtration is important for removing suspended particles from wastewater effluents penetrating the soils and for retaining the microorganisms that facilitate biological decomposition of dissolved and suspended organic matter. Even though the removal of suspended particles from water flowing through soils is easily observed, the processes involved are difficult to describe in simple cases. Listed below are three of the simplest mechanisms which might be combined to describe more complex situations.

Case I—Straining at the soil surface. Under these conditions the suspended particles accumulate on the soil surface and become a part of the filter.

Case II—Bridging. Under these conditions suspended particles penetrate the soil surface until they reach a pore opening that stops their passage.

Case III—Straining and sedimentation. This includes all of the conditions for Case I and Case II except that the suspended particles are finer than half of the smallest pore openings.

Irrigation with wastewater has a marked influence on the chemical equilibria in the soil. Organic matter and clay added in the suspended solids can increase the cation exchange capacity of the soil (Ramati and Mor, 1966). Many of the dissolved chemicals in wastewater influence the suitability of the soil for crop production. Nitrogen and phosphorus compounds have a beneficial fertilizer value when retained in the soil and utilized by the crops. Pollution of groundwater by nitrates which move freely in the soil can be a serious problem (Groundwater Contamination, 1961; Stewart, 1967). Release of phosphorus from soils to our surface waters can also contribute to pollution (Taylor, 1967). Boron content has caused concern in areas where boron-sensitive crops are irrigated with wastewater (State Water Pollution Board of California, 1955). Toxic concentrations of copper and zinc have apparently accumulated in the soil at sewage farms (Rohde, 1962).

The application of soil mantle disposal to upgrading lagoon effluent is limited by soil and groundwater characteristics. However, most lagoons are constructed in areas where land is available and thus capital investment for land mantle disposal is relatively low. Further, soil mantle disposal does not create a sludge disposal problem, has a low maintenance and operation cost, and may provide additional irrigation water in arid regions. Therefore, it is felt that soil mantle disposal may be of practical value in upgrading lagoon effluent.

Soil Mantle Disposal
Experimental Design

The experimental design for use of the soil mantle as an algal cell removal process consists of several interacting processes. The first rationale for a process would be that the soil mantle system would essentially be used as a storage compartment for the effluent. There would be no surface runoff or groundwater movement of the effluent. Evaporation from the soil, hydraulic conductivity, and lateral dispersion would determine the storage capacity of the soil. This process would have several severe disadvantages, such as seasonal variation in soil moisture storage potential, large volume of soil necessary, and extensive drainage and surface leveling to minimize runoff and groundwater implication.

Another process design would be to consider the water requirements of the vegetation being irrigated and the possible removal of the algal cells by impaction and impingement of the cells on the plants.

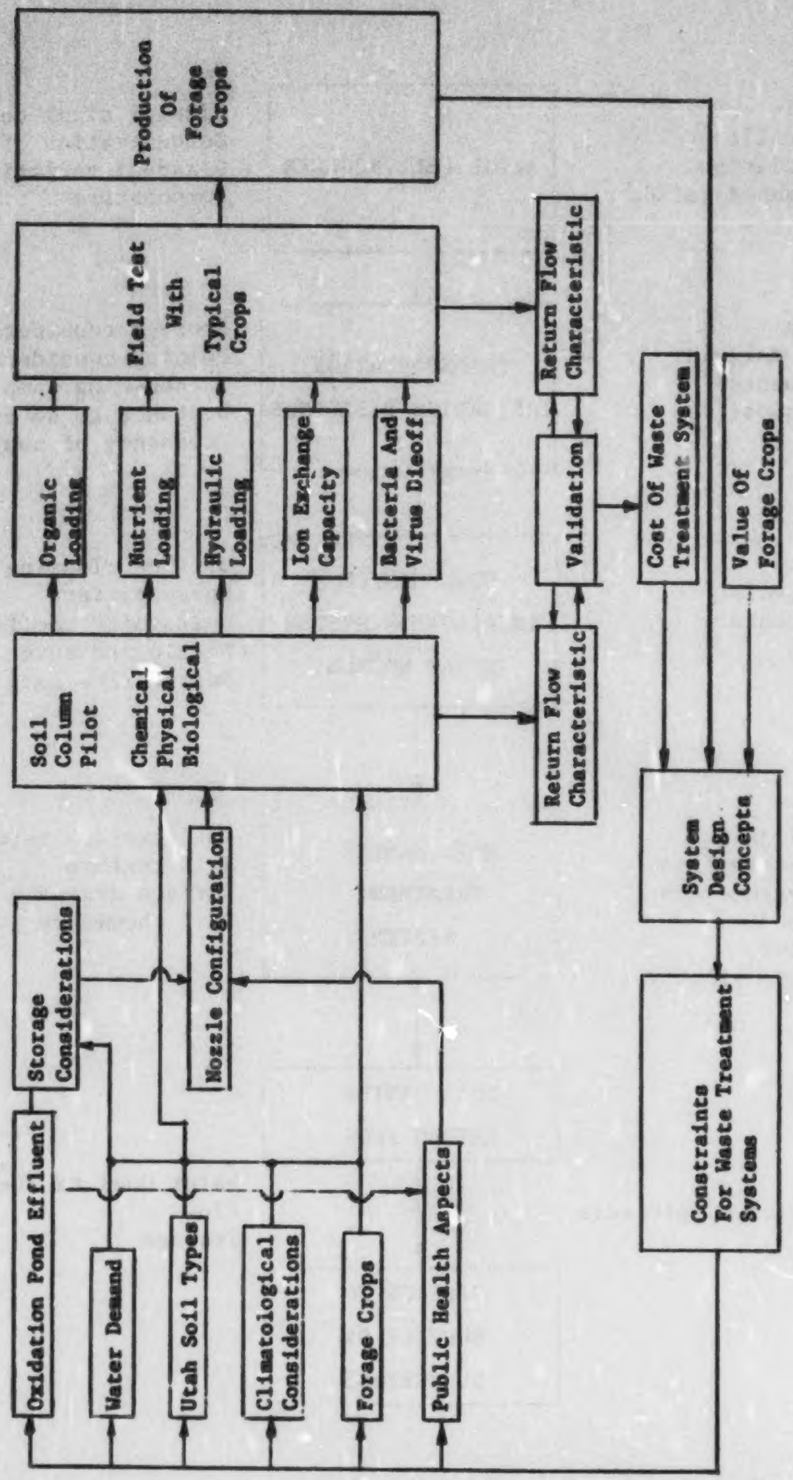

Oxidation Pond Effluent
Water Demand
Utah Soil Types
Climatological Considerations
Forage Crops
Public Health Aspects
Storage Considerations
Nozzle Configuration
Soil Column Pilot
Chemical Physical Biological
Organic Loading
Nutrient Loading
Hydraulic Loading
Ion Exchange Capacity
Bacteria And Virus Dieoff
Field Test With Typical Crops
Production Of Forage Crops
Return Flow Characteristic
Validation
Return Flow Characteristic
Cost Of Waste Treatment System
Value Of Forage Crops
System Design Concepts
Constraints For Waste Treatment Systems

Figure 1. Information flow chart for the research project.

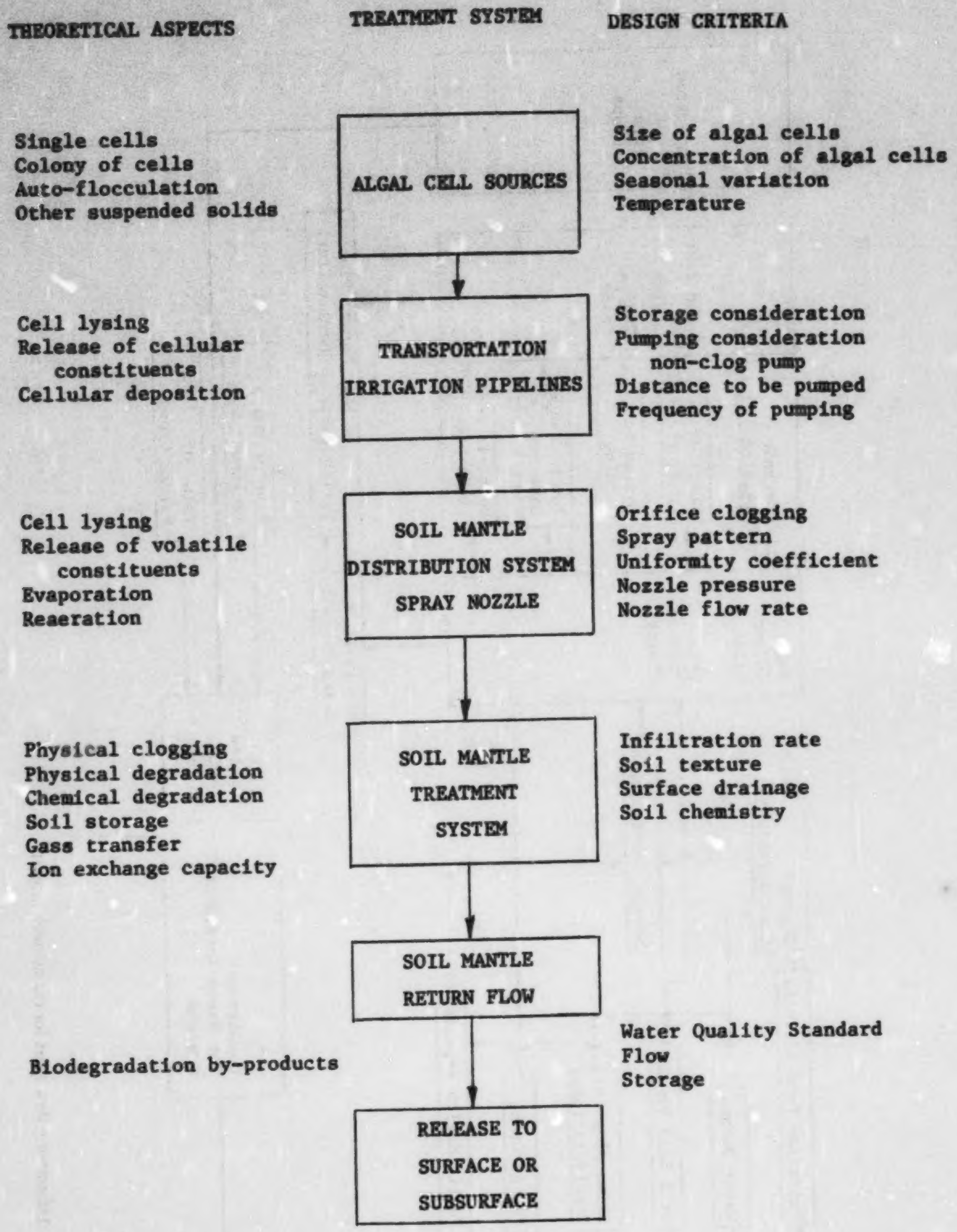

THEORETICAL ASPECTS
TREATMENT SYSTEM
DESIGN CRITERIA
Single cells
Colony of cells
Auto-flocculation
Other suspended solids
ALGAL CELL SOURCES
Size of algal cells
Concentration of algal cells
Seasonal variation
Temperature
Cell lysing
Release of cellular constituents
Cellular deposition
TRANSPORTATION IRRIGATION PIPELINES
Storage consideration
Pumping consideration non-clog pump
Distance to be pumped
Frequency of pumping
Cell lysing
Release of volatile constituents
Evaporation
Reaeration
SOIL MANTLE DISTRIBUTION SYSTEM SPRAY NOZZLE
Orifice clogging
Spray pattern
Uniformity coefficient
Nozzle pressure
Nozzle flow rate
Physical clogging
Physical degradation
Chemical degradation
Soil storage
Gass transfer
Ion exchange capacity
SOIL MANTLE TREATMENT SYSTEM
Infiltration rate
Soil texture
Surface drainage
Soil chemistry
SOIL MANTLE RETURN FLOW
Water Quality Standard
Flow
Storage
Biodegradation by-products
RELEASE TO SURFACE OR SUBSURFACE

Figure 2. Soil mantle.

The root zone depth could be the level of intermittent saturation required to satisfy the plants. This process design has several limitations also, one being the seasonal productivity and the amount of water necessary for optimum plant growth.

A third process design could be envisioned that would incorporate the advantages of two previously mentioned systems. Soil moisture storage as well as vegetation water requirements could be considered on a seasonal basis. During the non-productivity period of the year, the soil mantle system could be loaded to maximum soil moisture conditions while during the growing season the water requirement of the vegetation controls the rate of application. A storage capacity of 120 days would suffice for cold weather operations where ground conditions would not allow irrigation.

The soil only system would develop information typical of spring and fall spray irrigation of oxidation pond effluent. Meaningful data concerning soil moisture and storage capacity in terms of application rates and cell removal could be developed. The vegetation plot would allow an analysis of the transpiration rates and the effect of plants on the filtering phenomena of algal cells, such as impingement, impaction, etc.

Soil mantle facility

The Utah State University reclamation farm (110 acres) is located approximately in the center of Cache Valley, where the land is essentially flat. The slope is actually 0.07 percent to the west. Most any type of irrigation system can be used without expensive leveling. Much of the land in this area contains humics which makes efficient farming extremely difficult. Fortunately only about 3 acres are classed as humicy on the reclamation farm.

The top soil over most of the farm is extremely thin (1 to 2 feet). The texture of this material is mainly silty clay loam. Below this top soil is a tight impermeable clay which may be gleyed or mottled depending on the location on the farm. A soil survey has been conducted on the reclamation farm by drilling 34 test holes cored to a depth of 11 to 12 feet. The majority of the soil is clay with silty clay, and silty-clay-loam within the surface 2 to 3 feet. Most of the clay is tight and mottled or gleyed. A stratified sand, silty clay layer can be found under most of the farm at a depth of from 10 to 12 feet. This material may be helpful in removing the water coming up from the aquifer and down from the surface.

An open drain exists on the reclamation farm to remove surface water and some groundwater. During the winter and spring months the drain carries off about 3 second feet, helping drain the surface of about 400 acres on the immediate area of the farm.

Experimental design

The experimental design will consist of eight 50 x 50 foot test plots irrigated at three different rates (2, 7, 14 in/week). The effluent from Logan's oxidation pond will be pumped approximately 1/2 of a mile to the reclamation farm. Two test plots for each application rate consisting of a forage crop, alfalfa, and a soil only treatment plot will be irrigated 6 months out of the year for 2 years. One 4 in. drain pipe will collect the return flow at a depth of 3 feet. A sample port outside of the plot will allow the collection and subsequent analysis of the return flow. It is anticipated that only 14 inches/week will saturate the soil to the point of producing a return flow on a continuous basis.

A slotted two inch soil corning device will be used to obtain samples on the various plots on a weekly basis. The slotted corer will allow analysis of the vertical stratification of the soil and its associated water quality parameters. The upper most section of the soil mantle will be most actively involved in removal of the algal cells. The other water quality parameters, BOD_5, N-forms, P-forms, and dissolved organic carbon will be analyzed on a vertical basis to predict return flow characteristics.

Ten random samples will be obtained every two weeks from the nine test plots. The following test will be performed on the stratified samples over the two 6-month test periods.

	Forage Crop	Barren Soil	Control
Organic Content	10/twice monthly	10/twice monthly	10/twice monthly
N-Forms (NH_4, NO_3)	10	10	10
P-Forms (Total, dissolved)	10	10	10
Soil moisture	10	10	10
BOD_5 (If saturated)	-	-	-
	40	40	40

The proposed method of application of oxidation pond effluent is through a solid set irrigation system. The system will be completely automated, controlled by electric valves which will be varied to give irrigation in blocks of four laterals operating simultaneously. The system will be designed such that the time of irrigation of any one block can be left from a few minutes up to several hours, and the

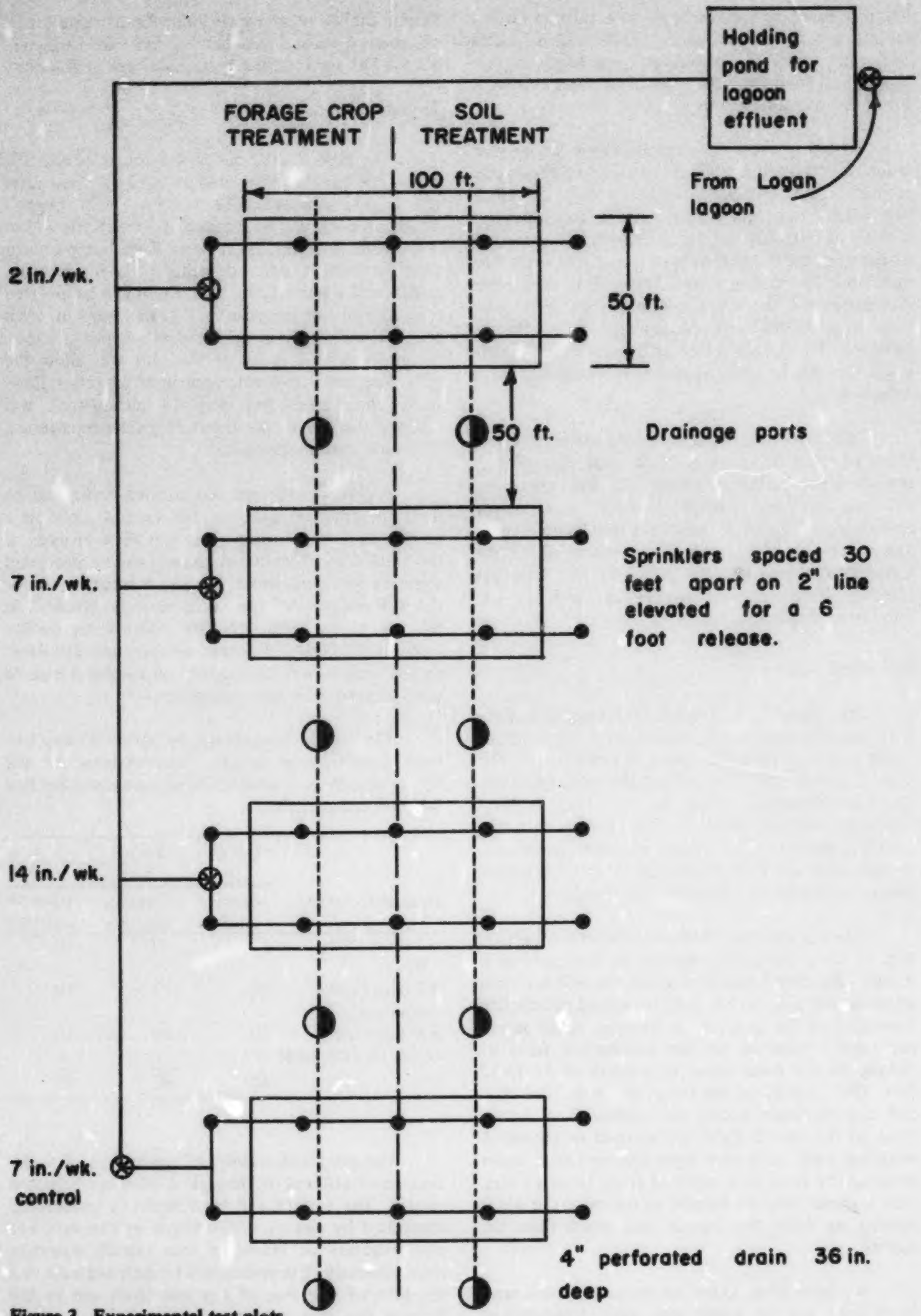

Holding pond for lagoon effluent
From Logan lagoon
FORAGE CROP TREATMENT
SOIL TREATMENT
100 ft.
50 ft.
50 ft.
Drainage ports
2 in./wk.
7 in./wk.
14 in./wk.
7 in./wk. control
Sprinklers spaced 30 feet apart on 2" line elevated for a 6 foot release.
4" perforated drain 36 in. deep

Figure 3. Experimental test plots.

Table 1. Soil analysis for lagoon effluent irrigation study.

	Drainage Farm Soil	Hyde Park Bench	Brigham City Bench	Four Miles South Main Logan
Texture	clay	silt loam	sandy loam	silt loam
Organic Matter %	5.5	1.9	2.3	3.7
pH	8.1	7.6	7.1	7.4
EC_e mmhos/cm	0.9	0.6	1.1	0.5
Phosphorus, ppm	7.1	4.5	13.0	27.0
Potassium, ppm	490	398	171	490
Lime	++	++	+	+
Exchangeable Sodium, me/100 g	0.8	0.2	0.2	0.3
Total Sodium, me/100 g	1.2	0.2	0.2	0.3
Water Soluble Sodium, me/100 g	0.3	< 0.1	< 0.1	< 0.1
Cation Exchange Capacity, me/100 g	19.7	17.7	9.9	23.6
Percent Saturation	83	42	28	56

system will automatically sequence from one block to the next. Any one block can be completely skipped in an irrigation sequence.

The sprinkler systems will be designed to deliver 33 gpm/test plot. The sprinkler heads will deliver .26 in/hour operating at 55 lbs/in^2 with a uniformity coefficient of above 85 percent. To achieve the desired application rates of 2 in/week, 7 in/week, and 14 in/week at weekly irrigation periods of 8, 24, 48 hours respectively would be required. This would use a total of 115 hours a week which would allow a simple timing control to operate off of only one supply system.

Objective

To determine the efficiency of algal cell removal from oxidation pond effluents spray irrigated on one type of vegetation and on soil only control plot and an effluent with no-algal cell control plot:

1. Design a drainage system for a one acre experimental plot to monitor the effect of the soil mantle system has on algal cell application.

2. Design a solid set irrigation system to deliver three flow rates on three different experimental plots as well as a control plot.

3. Install the drainage and irrigation system and the sampling and flow measuring devices.

4. Spray irrigate the oxidation pond effluent on the experimental plots measuring the following:

   a. Initial algal cell concentration, BOD, suspended solids, N-forms, and P-forms.

   b. Continuous measurement of irrigation rates, soil moisture, drainage return flow, and evaporation rate.

   c. Analyze drainage return flow for algal cells, suspended solids, BOD, N-forms, and P-forms.

   d. Measure the productivity of the two vegetation types at the three application rates and with the no-algal cell effluent irrigant.

   e. Repeat the above experiment after the first growing season in the fall and the spring.

   f. Determine the soil storage capacity as a function of algal cell removal; determine the application rate as a function of algal cell removal; determine the vegetation as a function of algal cell removal; determine the seasonal effect of algal cell removal by the various experimental plots.

   g. Determine the system design based upon cost of equipment, operating cost, and any economic benefit from increased production.

Introduction

Nature of the problem

During the last few years, people have become increasingly concerned about the deteriorating quality of the environment. This concern is beginning to express itself in many ways. Citizen action groups have formed, laws are being passed, and more and more research is being conducted in order to solve these problems.

Among the laws being passed in the State of Utah is a law that will increase the water quality standards of most of the receiving waters in Utah from Class D to Class C. As a result, the effluent discharged into any of these receiving waters must meet a more rigid standard of quality. The benefits of such legislation should include more aesthetically pleasing water and better environments for wildlife. Obviously these improvements are extremely desirable and in general will be welcomed.

There are however, certain rather important and critical considerations that arise from such a course of action. In many cases wastewater treatment facilities have already been constructed but were only designed to meet Class D standards. When the Class C standard becomes law, these existing facilities will no longer be adequate and therefore will no longer be allowed to discharge their effluents into any receiving water covered by the law. The implication is clear. Either existing facilities must be improved to the point where the effluent can meet the new receiving water standard, or a different method of disposing of the effluent must be found. Improving present facilities is going to be costly. At the hearings held in Logan, Utah, in August of 1971, one industrial representative asserted that the cost to his company to meet the new standard would be in the neighborhood of one million dollars. Sums of this magnitude represent severe financial burdens for large communities as well as industry. This figure becomes even more alarming when initial capital costs are considered. But an even more critical situation is the small community with a population of a few thousand or less. The sums of money required will probably be less, but still will be considerable. With rather small tax bases, the large capital investment required for treatment facilities that can meet the standard is for all practical purposes out of the question. When considered on a per capita basis, the problem becomes even more clear. Given a tax burden of one million dollars, a community of one thousand taxpayers would each be required to pay one thousand dollars. In a city of one hundred thousand people, each taxpayer would only be required to pay ten dollars. Therefore, the practice of discharging treated waste effluent into the receiving waters is no longer possible for these communities. Obviously the problem becomes one of finding an alternative method of disposal.

Purpose and scope of the study

The objective of this investigation is to study the wastewater stabilization pond and the possible use of the effluent as an irrigation water. Typical pond effluents will be examined for possible toxic effects of constituents and also for possible beneficial effects such as fertilizer value.

Consideration will also be given to the application of land use planning techniques for improving stabilization pond system appearance and utilization. Many pond systems perform rather well from an engineering point of view, but appear as no more than well engineered "holes-in-the-ground" visually. Suggestions will be made for improving the appearance of these ponds such that they improve rather than blight the landscape. These suggestions will be supplemented with examples of actual projects to indicate just a few of the possibilities available.

In summary the specific objectives to be dealt with in this study are as follows:

1. Investigate the limitations of pond effluent for use as irrigation water for crops.

2. Study the use of land planning techniques to make ponds more visually pleasing.

3. From the above studies, make suggestions for alternatives to meet the stated problems.

4. Suggest possible research activities necessary.

Utah Water Quality Standards

Upgrading from Class D to Class C standards

The difference between Class D and Class C effluent quality standards can be compared by referring to Table 2 (Middlebrooks, 1971). The most significant difference between the two standards is the 5-day BOD values. The reduction from 25 mg/l to 5 mg/l is the parameter that causes the greatest concern. Middlebrooks et al. (1971) have shown that present systems including waste stabilization ponds do not meet Class C standards in this regard. It is this aspect of the Class C water quality standards that necessitates the rather expensive modifications alluded to in the Introduction.

The other parameter of interest is the specification of fecal coliform levels separately in the Class C standard but not in the Class D standard. Middlebrooks et al. (1971) have commented however, that if a plant by proper operation is meeting the Class D coliform standard, in all likelihood it will also meet the Class C standard. Therefore the revised coliform standard should not in general reflect itself in increased capital cost improvements.

Table 2. Specific standards established for Class "C" and "D" water quality standards pertaining to wastewater treatment plant effluents.

Parameter	Concentration or Units	
	Class "C"	Class "D"
pH	6.5 - 8.5	6.5 - 9.0
Coliform, monthly arithmetical mean	5,000/100 ml	5,000/100 ml
Focal coliform, monthly arithmetical mean	2,000/100 ml	-
BOD_5, monthly arithmetical mean	5 mg/l	25 mg/l
Dissolved oxygen	5.5 mg/l	-
Chemical and radiological	PHS drinking water standards	PHS drinking water standards

An important implication of changing from Class D standards to Class C standards is the improved efficiency required of the existing plant. If it is operating very inefficiently, the new facility will either be overdesigned resulting in even larger capital costs for improvements or it will be overloaded and will not perform as required. The improved efficiency is again likely to be an expense that would be difficult for a small community to sustain.

The standards discussed above in effect apply only to plants actually discharging effluents to receiving waters. In other words, the change is not a change of effluent standard but rather a change of stream standard. Herein lies the possibility of using stabilization pond effluents for irrigation. If the standard was strictly an effluent standard, systems would not be allowed to discharge their effluents at all and thus total containment would be the only alternative.

Waste Stabilization Pond Considerations

Economics

One of the most appealing aspects of stabilization ponds is the relatively low capital cost required. In Figure 4, a comparison of construction costs for stabilization ponds relative to the construction costs of several other types of conventional sewage treatment facilities is shown (Economy in Sewerage Design and Construction, 1962). The data were compiled by the State of New York, but the figures are generally consistent with other geographical areas. Middlebrooks et al. (1971) performed similar investigations for several areas which are consistent with those compiled by New York. Clearly the waste stabilization pond is much less expensive. Note in Figure 4 that the cost of the land is not included. Since pond systems usually require larger parcels of land, the actual difference would be somewhat less.

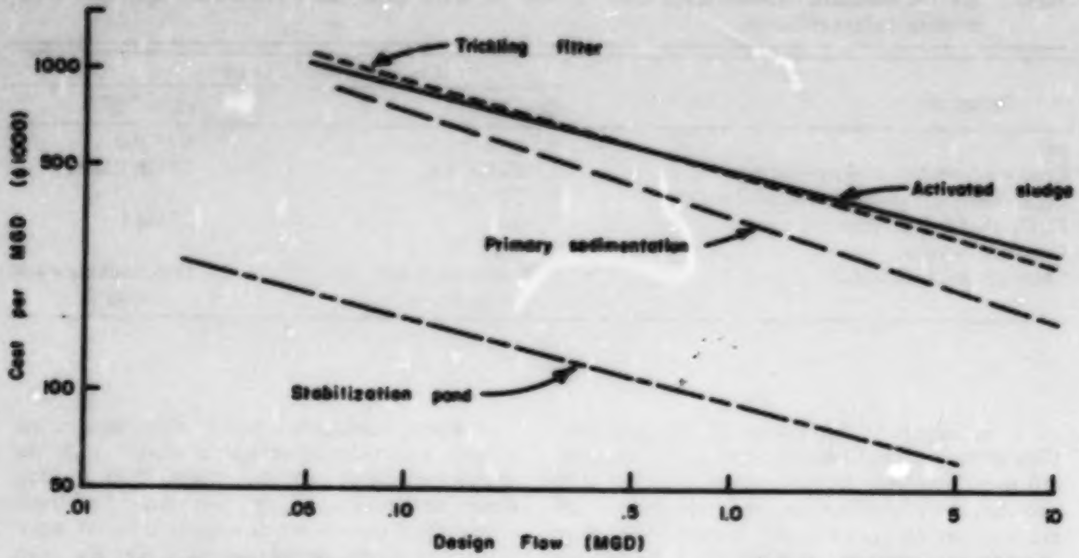
Waste stabilization ponds offer another important economic advantage associated with the operational aspects of the system. With trickling filters or activated sludge one has a relatively sophisticated process which requires constant supervision and highly skilled operators. On the other hand, a waste stabilization pond from an operations point of view is not as sophisticated and so the supervision requirement and the level of skill needed is subsequently lower. Both of these advantages will manifest themselves as a savings in operation cost.

To put these cost figures into proper perspective, one must think about the situations for which this investigation is intended. If a small town is considering changing over from a septic tank system to some type of central municipal system, clearly the waste stabilization pond is the answer economically. But as was pointed out in the section concerning water quality standards, the Class C standard cannot be met. Perhaps even more perplexing is the situation where the community already has gone to the expense of constructing a pond system and must spend more money on a more sophisticated system in order to meet the new standard. Obviously the need to meet the standard stands in conflict with the ability to raise the necessary funds.

Since the economics of the situation dictate against more advanced treatment systems, some alternative must be sought. The alternative which seems rather appealing is irrigation.

Design

Since a great number of articles and papers have been devoted to this subject, only a brief description will be given here. Waste stabilization pond depth, detention time, shape, mixing, and loading will be discussed. These engineering aspects will be coordinated with site planning considerations in the design of a waste stabilization pond system.



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Curves represent construction costs only and do not include land, engineering, legal or other administrative costs.

Figure 4. Sewage treatment plant construction costs.

The most significant effect of depth on the ecology of the stabilization pond is the decrease in light intensity with increased depth below the water surface. Oswald et al. (1964) has shown that in order for a pond to function aerobically throughout its depth, the pond depth must be less than 1 foot. Obviously, in general, this would be impractical because the land requirement would become enormous. Waste stabilization ponds, as most frequently employed, range in depths between 3 and 7 feet (Jones et al., 1971). Since light intensity penetrating to these depths in most wastes is insufficient to support photosynthetic activity, the lower regions of these ponds are usually devoid of oxygen. These ponds are referred to as facultative ponds. Another aspect of pond depth that is important in the operation and performance of the pond is the control of emergent vegetation such as cattails and bulrushes. This consideration suggests pond depths of at least 3 feet. Control of emergent vegetation is an important step in controlling insects such as mosquitoes (Kitterle and Enns, 1968). Another factor which must be considered in determining the depth of the pond is the water table. The cooling effects of groundwater and the possibility of contaminating the groundwater require that the pond bottoms be above the local water table.

Pond detention times are important to allow sufficient exposure of the waste to the biological forces necessary to treat the waste. In conventionally operated overflow type waste stabilization ponds, the detention time generally ranges between 25 days and 3 months. Many ponds are operated on a non-overflow basis so that inflow to the pond is balanced by seepage and evaporation. To avoid contamination of groundwater aquifers by ponds containing toxic chemicals, it is best to minimize seepage. Detention times are affected by temperature effects on biological activity and so need to be longer in winter than in summer.

Complete agreement as to the most effective shape for waste stabilization ponds is not to be found in the literature. One reason for this lack of definition is that a pond is both a hydraulic system and a biological system. A number of investigators have argued that hydraulically, ponds are most efficient in a two-to-one rectangular shape. Others have stated that the optimum shape is round or square. Still others feel that the shape is not nearly so restricted in terms of overall performance. All agree, however, that the cells should be regular in shape with no isolated or partially isolated areas where circulation might be impeded, weed growth encouraged, etc. (Jones et al., 1971).

Most authorities agree that multi-cell systems are more successful than single cells. The principal reasons for this are: (1) reduced tendency for short circuiting, (2) less bank erosion caused by wave action, and (3) more flexible operation (Jones et al., 1971). They also produce waters of lower coliform counts. Marais (1972) has done considerable work in developing multi-cell theory. He found that single pond systems were not as effective as multi-cell systems in reducing bacterial counts. His calculations show that for 90 percent removal, one pond is sufficient; for 99 percent removal, two ponds are needed; for 99.9 percent, three ponds; for 99.99 percent, four ponds, and so on.

There are two points of view concerning the use and value of mixing. Pipes (1961) feels that a pond operates most satisfactorily if it is kept completely aerobic. Marais (1972) has shown that bacterial die-away is greater in ponds where gentle mixing occurs. He points out that an aerobic environment is crucial in maintaining a high rate of die-away of the fecal organisms. The rate of die-off diminishes significantly when conditions become anaerobic. He feels that with ponds in general, wind and temperature cause a gentle mixing and lead to the proper conditions. In those cases where mixing does not occur, Marais suggests some type of stirring mechanism. The reason for the mixing is to induce algal growth which in turn leads to greater oxygen production. The other point of view, held by Oswald et al. (1964) among others feel that both aerobic and anaerobic conditions are necessary for best operation. When mixing occurs, oxygen is sent into the anaerobic region. Methane bacteria which perform an important part of the anaerobic process are very sensitive to oxygen and are thus unable to function.

Another aspect of mixing is the contribution from wind. Initially wind was considered useful because it increased the effective surface area and so allowed greater aeration. More recent thinking (Jones et al., 1971) tends to look upon wind with disfavor because of several reasons: (1) The question of the value of mixing, (2) the insignificant amount of aeration that actually occurs, and (3) the loss of water quality because of increased evaporation rates with associated increased chemical concentrations. Wind can also cause increased soil bank erosion by wave action and disperse odors into populated areas. In view of these disadvantages, sites now are recommended that have some sort of screening to reduce wind effects.

Pond loading is one of the key technical aspects of the design. The total pond area required to treat the waste of a given community is determined on the basis of organic loading in most cases, usually reported in terms of pounds of BOD per acre per day. Oswald et al. (1964) give the maximum load on facultative ponds as 60 pounds BOD per acre per day. However, the values most frequently recommended fall in the range of 20 to 50 pounds BOD per acre per day. It is obvious that when a pond has been overloaded, its operating efficiency is reduced. It may not be obvious however, that underloading can also hinder performance. Herman and Gloyna (1958) among others have pointed to low organic loadings as the cause of unsatisfactory pond performance. Some recent research (Assenzo and Reid, 1966; Meenaghan and Alley, 1963) has thrown suspicion on the surface area loading criteria. It indicates that a volume loading criteria would be more rational, particularly in the case of facultative and anaerobic ponds.

Land Use Planning Considerations

Introduction

Too often in the past designs for waste stabilization pond systems have neglected an important aspect of the design, site planning. The consequences of such an approach are installations that, despite their technical performance, are unsightly. Not only is the appearance undesirable, but important natural design elements that could markedly improve the overall design are not taken into consideration. Clearly the best design can only be achieved by becoming familiar with all design tools available whether they come from the area of engineering or the related fields of site planning and landscape architecture.

This chapter will present an inventory of a few of the more important tools available from the field of landscape architecture. Because the process of site planning has no best approach, no attempt will be made to describe a step by step process. Such things as plant materials and wildlife will be shown to offer great potential in solving certain problems inherent to waste stabilization pond installations. Only a few of the potential planning concepts will be mentioned. No attempt will be made to list all of these things or to explore their usage in great depth. Some of the considerations in site selection will be given when planning a pond installation. Two examples of projects that were designed with the inclusion of planning concepts will conclude the section.

Site selection considerations

Site planning is based on two primary considerations. First, the nature of the site, its character and its distinguishing and unique features must be considered. Second, consideration must be given to the human purpose for which the site is to be used. These two considerations are linked together in a circular way that is rather interesting (Lynch, 1962). The purpose for which the site is to be used will influence the interpretation of the inherent characteristics of the site. At the same time, the inherent characteristics of the site determine the purpose for which the site may be used. As one can see, these two considerations must be merged together in some manner in order to optimize the use of the site. Failure to successfully merge these two considerations will result in a system aesthetically displeasing and possibly substandard in engineering performance.

The primary purpose of the waste stabilization pond for this study, besides the obvious treatment aspect, is to provide a source of irrigation water. This allows effective disposal of the effluent and at the same time offers an agricultural benefit by possibly reducing costs. In terms of planning, a number of implications arise. If agriculturists are going to use the water, the pond system site may need to be centrally located in order to provide the most effective service to the users. Future land use projections are important in this regard so that today's genius does not become tomorrow's folley.

A centrally located site may not be the best site. Certain locations may require costly distribution systems and added pumping costs. It may be that by using an alternative site not centrally located, some natural terrain feature such as a hill or a deep ravine may be avoided with a resultant savings in cost. This saving should be compatible with the need for efficient service to the individual user.

Another important consideration is the pumping requirement dictated by the site. If the pond system is built in the low point of a valley, gravity flow can be used to transport the waste to the stabilization pond. However, with the ponds at the lowest point, effluent distribution will then require extensive pumping unless the users are close to the pump (assuming there is no abrupt change in topography). One would generally tend to regard such a situation as unlikely because low points in valleys are often swampy and are not usually suitable for agriculture. Where exceptions do occur, the alternatives for decision making are expanded.

Problems amenable to planning techniques

The problem of odors could become rather significant if the pond system was near a residential area. If a pond system was situated in a low valley area, the problem would be still greater. The odors would be harder to diffuse because of the cool damp air which would tend to hang in a depression of this type. In general, the literature asserts that when ponds are properly designed, odor problems do not occur. If odors do become a problem, certain chemical treatments such as chlorination, aeration, and additional nutrients are commonly used. Planning techniques can also be useful in overcoming any odor problems which might occur.

The concept of pond mixing by wind action can also present problems. Bank erosion due to excessive wave action is an important consideration in this regard. On the other hand, Marais (1972) felt that mixing was useful and that sun and wind were important contributors to this mixing.

Mosquitoes and other insects are significant problems. This is particularly so if some type of recreational area or activity is planned as part of the pond system design. Control of emergent vegetation in the ponds has been cited as the major deterrent to these pests. Even so, chemical treatments (DDT for example) are still apparently necessary. Planning techniques can offer some rather interesting approaches to this problem.

Another problem is the type of site commonly selected. Frequently sites are chosen where the soil condition is poor and not amenable to most plants. This adds to the unsightly appearance of the pond system since plants and trees commonly used for beautification will not survive in such unfavorable conditions. Again by utilizing the talents of the landscape architect, this situation can be successfully approached.

The water table depth must be ascertained. If the water table is high, precautions must be taken to protect the groundwater from contamination and to prevent seepage into the pond system and affect the designed loading rate. The depth of the water table may also influence the type of vegetation that one may want to introduce.

The topography also is an important consideration not only for the obvious reason of reducing construction costs on smoother land but for potential advantages of modifying the microclimate. Steep slopes are of course undesirable but flat ground may not be the best answer either. A careful study of the individual situation must answer that question.

The stated considerations do not constitute all the elements that require the designer's attention, but do indicate the importance of this step. It should also be clear that the engineer can reach a better conclusion by collaborating with a landscape architect who is trained in the field of site analysis.

Planning techniques

An existing berm may lend itself to the overall design. It may be used for modification of the wind effects by acting as a screen or wind break. By using it as a buffer, the wind can be modified so that wave action in a stabilization pond is reduced with an associated reduction in bank erosion. A berm could be used as a deflector to encourage mixing and so encourage the aerobic conditions recommended by Marais (1972). The berm could be used as a visual screen to cover objectionable aspects of the installation. Berms might also assist in the distribution of sewage flow from pond to pond by providing different levels for series ponds to be built and therefore allow gravity flow. The design of natural and man-made berms in this distribution can be best handled with a collaboration of engineer and landscape architect to optimize performance and appearance.

The berm may lend itself nicely if some recreational activity is desired. A park's beauty can be enhanced by recongizing the potential and then allowing the landscape architect to use his talents to maximize its effect. Nearby hills may also function as game preserves and thus attract wildlife. This in turn can add to the beauty of the surroundings which will make the area more attractive if a park is intended. Obviously a berm may represent a vital parameter and before simply bulldozing it flat, a competent landscape architect should evaluate its potential.

The use of existing and induced plant materials can provide a great number of advantages. By using existing plant materials in combination with judicious induced planting, the general public can be discouraged from entering certain areas reserved for maintenance personnel and encouraged to use the areas set aside for the public. Trees can be used to screen off areas from view which are unsightly. Conversely if certain views are desirable, trees can be used to frame these sights and thus bring emphasis to the sight for the viewer. Trees are effective as wind breaks and are even more effective for wind modification than are berms. In a study conducted at the Kansas Agricultural Experiment Station (Olgyay, 1963), trees were shown to reduce wind effects over a much greater distance beyond the wind break than solid wind breaks. The solid type was able to provide greater close-in reductions but velocities returned to original values sooner.

Correct usage of trees can modify and reduce odor problems. Certain trees such as the black locust and the tree of heaven emit aromas that can markedly reduce offensive odors. The mechanism is dilution and is effective. Flowers can also be useful in this same regard.

Trees can also be used for space definition. People are rather clearly affected emotionally by the type of space they find themselves in (Simonds, 1961). If the space is high and majestic, people might be affected with the sense of feeling small and overpowered. If the space has a low overhead and is not large, the feeling could be intimate and conducive to fellowship and small social gatherings such as family picnics. If a park is to be a part of the design these factors become important in terms of human scale and desired response. Frequently one sees an area that appears pleasing to the eye but no one uses the area. One possible explanation is that the space relationship of the area makes people feel uncomfortable. Only through the skillful use of these concepts will such an area really be successful.

The use of trees and other plant materials for attracting specific birds that consume insects can provide a distinct benefit. Using birds in this way reduces insecticide requirements, saves money, and reduces the hazard of pesticide accumulations in the ecology. As was mentioned above, mosquitoes are largely controlled by preventing emergent growth. The mosquito population and other insect and fly problems can be further controlled by birds. Purple martins for example eat several times their body weight each day in insects. And since purple martins prefer lots of company, attracting this specie would provide a large number of birds which could almost virtually destroy any insect problems inherent to stabilization ponds. Wrens and bats among many others are also effective against insects. To utilize this very effective tool requires a knowledge of the kinds of plant materials or man-made structures (birdhouses for example) that will attract and keep these forms of wildlife (Davison, 1967; Stefferud, 1966). Again a competent landscape architect can offer assistance.

Examples of planning applications

After reviewing the discussion under Design, one should note that the use of shape for aesthetic design seems rather plausible. There are of course some limitations but perhaps something more organic than an ordinary rectangle is possible, as described by Oswald et al. (1967). The project was the construction of a waste stabilization pond system for a subdivision. The design was the integration of engineering and landscape architecture and resulted in a product vastly superior to a mere well engineered "hole-in-the-ground." The design is shown in Figure 5. Of significance are the pond cells which are not as stiff and formal as the common rectangle. The system also has been integrated into a park allowing for multiple use of the land. The designers have used many trees which meet the new thinking concerning reduced mixing for pond systems. The multi-cell design also offers improved treatment potential. The most important overriding conclusion with this example is the obvious compatibility between the two disciplines, engineering and landscape architecture. The increased cost for aesthetics results in a significant benefit for the citizens of that community. At the date of publication the project had not been constructed, but the ideas had been used in other projects. The authors call attention to one project, the Esparto ponds in California. They were interested in determining the possible odor problems since the system would be close to habitations and subsequently found that an odor nuisance did not result. They also found that visual and public health restrictions were also met.

Another project illustrating the use of good planning is the Santee project (Merrell et al., 1967). The Santee project is a total reclamation project

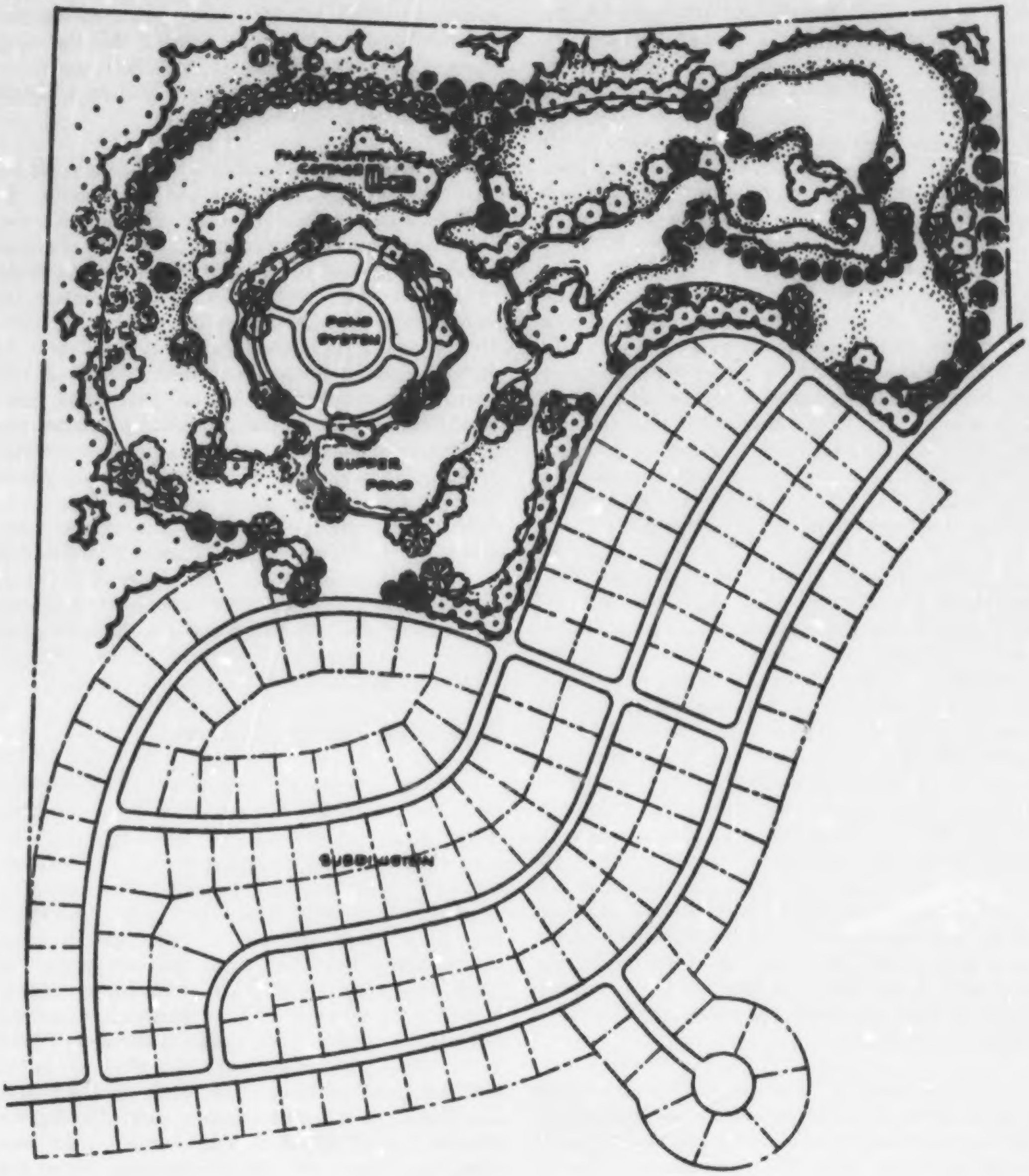

Figure 5. Pond system in park of subdivision design.

which includes several small reservoirs which serve as tertiary treatment. The project shows that a design can successfully include more than pure engineering. The Santee project includes areas for picnicing, boating, fishing, and even swimming. Here is multiple use of land with great variety resulting in benefits to a broad spectrum of people.

Whether one likes or dislikes the actual designs of the aforementioned projects is not as important as recognizing the feasibility of considering aesthetics in the overall design. The Santee project clearly shows the benefits of such planning by the number of people using the facilities. A good many users are tourists which of course brings an economic benefit to the community. This to some degree answers the question of greater project cost.

Based on the above considerations, the approach used by Oswald et al. (1967) in combining the talents of the engineer and the landscape architect seems desirable. This combination provides the sound technical basis for site planning and construction available from both disciplines plus the element of sharpened sensitivity to aesthetic values which is the most important tool of a competent landscape architect. Though such a combination clearly will increase the design cost, the long range benefits possible should be carefully considered before dispensing with planning considerations.

Effluent Components Considered for Irrigation

Introduction

Thus far in this discussion, the waste stabilization pond has been looked at in terms of economics and been found to offer distinct advantages over other types of conventional systems. Their operation and design has been studied and found to offer good wastewater treatment capability. Land use planning techniques were then discussed showing some of the considerations that can integrate the pond system into a community most effectively. In particular the land use concept of the pond system as a source of irrigation water was discussed. All of these considerations have shown the feasibility of the pond system as an irrigation source. The one remaining question is the chemical and bacteriological quality of the water.

Pertinent to the water quality discussion is the municipal use and its effect upon that quality. The chemical quality of the wastewater effluent is basically the sum of the original chemical quality and the municipal increment added. Conventional secondary treatment facilities only slightly affect the chemical quality. Their principal effect is in the biological area. The average municipal increments for a number of constituents are shown in Table 3 (U.S. Public Health Service, 1963). Specific industrial

Table 3. Average increments added by community use of water.[a]

Pollutants	Overall		Eastern		Western	
	Average	Range	Average	Range	Average	Range
BOD	22	8- 45	21	8- 33	23	10- 45
COD	111	36-218	96	36-159	218	218
ABS	6	2- 10	5	2- 10	8	6- 9
Na^+	69	8-115	59	42- 98	74	8-115
K^+	10	7- 15	9	7- 12	11	7- 15
NH_4^+	19	3- 50	21	9- 29	18	3- 50
Ca^{++}	17	1- 44	24	1- 44	13	2- 30
Mg^{++}	7	0- 24	8	3- 11	7	0- 24
Cl^-	75	14-200	53	14-102	92	20-200
NO_3^-	9	0- 26	11	0- 26	7	0- 15
NO_2^-	1	0.1- 2	0.8	0.1- 2	2	2
NCO_3^-	104	-44-265	147	49-265	81	-44-247
$CO_3^=$	-1	0- -10	-1	0- -5	-1	0- -10
$SO_4^=$	28	10- 57	28	12- 52	29	10- 57
$SiO_3^=$	16	13- 22	15	15	17	13- 22
PO_4^{-3} (total)	24	2- 50	19	2- 35	28	7- 50
PO_4^{-3} (ortho)	22	7- 34	20	12- 34	25	7- 34
Hardness ($CaCO_3$)	69	10-185	95	15-151	58	10-185
Alkalinity ($CaCO_3$)	85	-35-217	121	40-217	66	-36-203
TDS	323	128-541	287	128-457	352	194-541

[a] Source: U.S. Public Health Service (1963).

discharges into the community system could totally distort the value for a given component. Since generally communities will have a good source water, problems that may arise in using the effluents for irrigation are usually the result of the increments shown in Table 3.

The Utah State Division of Health has conducted extensive chemical quality tests on a great many waters of the state including several stabilization pond effluents. Data from these tests are summarized and presented in Tables 20 to 24 by the Utah State Division of Health (1968, 1969, 1970) in the Appendix. Based on the water quality factors tested by the Division of Health, an extensive literature review was performed to determine the effects of these constituents on the water quality for irrigation use. Beneficial as well as toxic effects are described.

In reviewing the literature it was discovered that a great many studies have been performed on the interactions of one component in the presence of a second. No attempt was made to include this information, except in a few cases, because of the great number of these papers.

Arsenic

Arsenic can be beneficial or toxic to plants depending on the concentration of arsenic and the specie of plant. Even though some beneficial results have been noted, arsenic is not presently considered an essential nutrient for plant survival and health. In toxic concentrations arsenic can cause rotting of roots, arrest seedling germination, and reduce seedling viability. Arsenic primarily settles in the root system resulting in significant plant growth reduction before it goes to top-growth. This means that a plant can be damaged without visible symptoms occurring. When physical damage is apparent, the damage seems to be the destruction of chlorophyll in the foliage. This condition is referred to as chlorosis and generally appears as yellowing of the plant leaves.

Several investigators have performed extensive investigations regarding the extent of arsenic in soils. Williams and Whetstone (1940) found in testing a wide variety of soils that natural arsenic occurred in concentrations from 0.38 ppm to 38.0 ppm. Moxon et al. (1944) found in studies of cretacious formations concentrations of 0.2 ppm to 64.4 ppm.

In studies conducted to determine toxicity levels, Vandecaveye et al. (1936) found that the soil samples from the top 6 inches of a number of unproductive fields contained from 4.5 to 12.5 ppm of readily soluble arsenic calculated as As_2O_3. Marked retardation of growth of young alfalfa and barley plants was noted. In other experiments, Lieberg et al. (1959) applied several concentrations of arsenic and found that at a concentration of 1 mg/l, either as arsenate or as arsenite, the growth of lemon trees was stimulated. However, when concentrations of 5 mg/l of arsenate or 10 mg/l of arsenite were applied, these concentrations were found to be toxic to top growth and root growth.

Bicarbonate and carbonate

Other than pH control, no evidence could be found that either carbonate or bicarbonate offers any beneficial effect at all. Evidence does exist however, that excessive amounts of these ions can be directly or indirectly toxic to plant growth.

Porter and Thorne (1955) found that sodium bicarbonate in substantial concentrations was indirectly responsible for iron chlorosis in beans and tomatoes. They used 10 meq/l sodium bicarbonate and observed a decrease in both chlorophyll content and growth compared to a concentration of only 0.3 meq/l. Arnon and Johnson (1942) found that by using lettuce, tomatoes, and bermuda grass, plants grew satisfactorily at pH values of 4 to 8 assuming plant nutrients were available. At pH values of 3 or less and 9 or greater, plant growth was reduced.

Harley and Lindner (1945) found that high concentrations of bicarbonate ion in irrigation water over a period of years produced a detrimental effect upon pear trees and apple trees. When the concentration was 3.44 meq/l to 5.0 meq/l, a marked decline in vigor was noted. A water with 1.5 meq/l showed no ill effects. Chlorosis was the predominant disorder in pears, but was only sporadic in apples. In addition lime concretions formed around roots and small carbon nodules formed on fibrous roots. These effects were entirely absent from trees irrigated with low bicarbonate concentrations.

In addition to chlorosis other undersirable effects from bicarbonates have been shown to occur. Steward and Preston (1941) found that large concentrations of potassium bicarbonate and dissolved carbon dioxide reduced protein synthesis and bromide accumulation by potato discs. At 20 meq/l of potassium bicarbonate, protein synthesis was almost totally arrested. Hassan and Overstreet (1952) found that increasing from 0 to 200 meq/l of bicarbonate ions reduced radish elongation. At 50 meq/l, elongation was reduced by 65 percent and at 200 meq/l, elongation was reduced by 98 percent.

Bicarbonates may also affect plants indirectly. One rather common effect is the reaction with available calcium ions to form calcium carbonate. The removal of available calcium acts to increase the sodium hazard by increasing the SAR value (refer to the section concerning sodium). Misra and Mishra (1969) found that carbonates, when added to a soil, reduce the availability of manganese in black, red, or alkali soils. Since manganese is an essential plant nutrient, this effect takes on some importance. The carbonates were shown to reduce the availability of manganese regardless of whether the carbonate was soluble or insoluble. (Refer to the section on the characteristics of manganese for more information.)

Wadleigh and Brown (1952) found that beans and Dallis grass are very sensitive to bicarbonate concentrations and that Rhodes grass and beets are relatively tolerant.

Studies have been made in order to specify tolerance limits for bicarbonates and carbonates in irrigation water. One such specification was based on the amount of residual sodium carbonate present. Residual sodium carbonate is defined as follows:

$$RSC = (CO_3^{=} + HCO_3^{-}) - (Ca^{++} + Mg^{++})$$

where the ion concentrations are in meq/l. Wilcox et al. (1954), in establishing limits for residual sodium carbonate, used Rhodes grass in sandy loam soil under greenhouse conditions. Waters containing more than 2.5 meq/l of residual sodium carbonate are unsuitable for irrigation over a long period without use of amendments, 1.25 to 2.5 meq/l is marginal, and less than 1.25 meq/l is probably safe.

Boron

Boron has for some time been acknowledged as one of the essential nutrients for plant growth. After the laying of much groundwork by early investigators, this fact was firmly accepted after the work of Sommer and Lipman (1926). Deficiencies of boron can cause any of several non-parasitic diseases. Some of these include top sickness of tobacco, heart-rot of sugar beets, cork disease of apple, brown rot of cauliflower, and raan of rutabaga.

Boron deficiencies will generally not occur if the water being used has a concentration of 0.10 to 0.20 ppm and is used in quantities of 1 acre-foot or more. A deficiency may occur under certain conditions in the presence of the calcium ion. Chapman and Vanselow (1955) showed that water low in boron but high in calcium will cause plants to exhibit an increased need for boron.

Table 4. Relative tolerance of plants to boron.

Tolerant	Semitolerant	Sensitive
4 mg/l	2 mg/l	1 mg/l
Athel	Sunflower	Pecan
Asparagus	Potato	Black walnut
Palm	Acala cotton	Persian walnut
Date palm	Pima cotton	Jerusalem artichoke
Sugar beet	Tomato	Navy bean
Mangel	Sweetpea	American elm
Garden beet	Radish	Plum
Alfalfa	Field pea	Pear
Gladiolus	Ragged robbin rose	Apple
Broadbean	Olive	Grape
Onion	Barley	Kadota fig
Turnip	Wheat	Persimmon
Cabbage	Corn	Cherry
Lettuce	Milo	Peach
Carrot	Oat	Apricot
2 mg/l	Zinnia	Thornless blackberry
	Pumpkin	Orange
	Bell pepper	Avocado
	Sweet potato	Grapefruit
	Lima bean	Lemon
	1 mg/l	0.5 mg/l

Source: Eaton (1939).

Boron may also be highly toxic to plants in excessive concentrations. Some plants, particularly citrus trees, are very sensitive and can be damaged by concentrations as low as 0.75 ppm. Table 4 is a table of plants and their tolerance to boron prepared by Eaton (1939). Notice that he classifies sensitive crops as those that will show slight to moderate injury at levels of 0.5 to 1.0 mg/l, semitolerant at levels of 1.0 to 2.0 mg/l, and tolerant crops at levels of 2.0 to 4.0 mg/l. In general past writers have left the impression that concentrations in excess of 4 or 5 ppm boron would cause serious damage to even the very tolerant plants. Investigators in the last 10 years however, have found that certain plants are much more tolerant than previously thought.

Oertli et al. (1961) performed a number of experiments with several varieties of turfgrass and found them to be very tolerant of boron. They subjected the plants to concentrations up to 10 ppm without any apparent reduction in growth. At 10 ppm, damage did occur at the leaf tips. This did not prove to be significant because the damage could be easily removed by frequent mowing.

In another study, Oertli and Roth (1969) studied the effects of boron on sugar beets, cotton, and soybeans. Their results showed sugar beets to be unusually tolerant. Even at concentrations of up to 25 ppm of boron, yields were not diminished. Applications of 40 ppm resulted in some leaf damage but yields were still rather good. Cotton showed a considerable tolerance for boron although less than sugar beets. Yields did not begin to drop off significantly until concentrations of 15 ppm were applied. Soybeans proved to be the most sensitive.

Yields were significantly affected at concentrations greater than 5.0 ppm. Notice however, that soybeans would be considered a tolerant crop using Eaton's (1939) criteria, Table 4.

Boron toxicity can also vary considerably depending on the stage of the plant. Langille and MaHoney (1959) conducted studies on oats and found that boron injury was quite dependent on the stage of growth. Boron was applied at the rate of 35 pounds per acre of borax ($Na_2B_4O_7 \cdot 10H_2O$) to a field of oats underseeded to a Ladino clover mixture. With the first noticeable oat growth, there appeared a definite chloratic condition of the oat seedlings on plots treated with boron. Affected seedlings showed a distinctly yellow and in some cases almost white appearance. The untreated areas appeared lush and green. However, as the plants began to mature, the injury began to decline rapidly. At the end of the season the average yields from the treated and untreated plots were 61.6 and 61.4 bushels per acre respectively. These yields indicate that the boron toxicity noted earlier in the season did not cause a reduction in yield under the conditions of the experiment. After examination of the plants and the soil, the authors suggest a minimum toxic level of 0.84 ppm of boron in the water. Concentrations of 0.84 ppm or less appeared to cause no apparent damage. This seems to disagree slightly from Eaton (1939) but the difference does not appear significant.

In a study using peach trees, McLarty and Woodbridge (1950) studied the effects of boron both in toxic and deficient amounts. When boron levels were deficient the peach trees showed significant die-back in the spring of twigs and branches on trees which had grown normally the previous season when boron levels were adequate. There was no warning this would occur. Leaf and flower buds appeared to form normally but failed to "break" normally when growth started in the spring.

When boron was in excess, small necrotic areas developed in the leaves along the midrib eventually resulting in perforations in the leaf. Small cankers sometimes also developed on the underside of the midrib and in the petiole. In more severe cases, cankers developed along the stems, tips of branches withered, and gumming occurred. Leaf symptoms like those described above were induced on 2-year old peach trees with the application of 5 ppm boron. The trees in general seemed to recover the following year when the excess boron application was discontinued.

Kelley et al. (1952) found that with carrots grown in sand cultures, boron toxicity symptoms occurred at 5 ppm for carrots grown in the summer and 10 ppm for carrots grown in the winter. Apparently boron toxicity in certain plants is quite sensitive to climate. Carotene content was independent of boron supply for all levels greater than needed to overcome deficiency.

Cadmium

Not a great deal has been presented on the potential toxicity problems related to plants. McKee and Wolfe (1963) suggest a tollerance limit of 0.005 mg/l for irrigation water on a long term basis. They based this value on two assumptions: (1) reported toxicity levels are correct (Lieber and Welsch, 1954) and (2) cadmium behaves like zinc relative to plant uptake and soil reactions.

In a study Schroeder and Balassa (1963) found that when vegetables normally devoid of cadmium were grown in soil heavily fertilized with 20 percent superphosphate ($H_2PO_4^-$), the plants absorbed cadmium. These plants included potatoes, string beans, beets, onions, peas, and carrots. They also found that vegetables normally containing cadmium absorbed larger quantities than normal. The plants were lettuce, turnips, radishes, and parsnips. The superphosphates were found to contain 7.25 ppm cadmium. The authors concluded that superphosphate can increase the cadmium uptake of plants and thereby reduce the plant's tolerance to cadmium.

In the same study, the authors applied a large amount of superphosphate to one of three plots of vegetables. Growth of the fertilized plot was far more lush than that of the two unfertilized plots. The plants used were peas, swiss chard, beets, and turnips. Schroeder and Balassa (1963) concluded that the plants were not damaged by 7.25 ppm cadmium.

Calcium

Calcium is another of the 16 basic elements required for plant growth. It is the dominant base of a "normal" soil. In addition to the benefits to plant growth and soil structure, calcium also offers benefits for biological activity. If soil conditions become either too acid or too alkaline, calcium will become deficient.

As pointed out in the section on boron, calcium can reduce the toxic effect of boron. Cooper et al. (1958) found that concentrations of calcium nitrate added to a water of 6.9 ppm of boron in amounts that increased the electrical conductivity of the saturation extract from 5.5 to 7.2 millimhos/cm reduced the usual boron effects by one-third. Calcium is then a useful amendment for reducing problems of boron toxicity.

The presence of calcium in the soil can cause potassium to be more readily available to plants.

York et al. (1954) showed that when lime and potassium fertilizer were used together growth yields were increased.

Excessive concentrations of calcium can also cause detrimental effects. These effects generally stem from calcium reacting with certain ions in the soil. For example, if calcium combines with carbonate, a buffering effect results. This in turn produces a decrease in nutrient availability. In addition, with calcium being precipitated as calcium carbonate, the sodium hazard increases (see section on sodium). Lund (1970) studied calcium and its effects in relation to magnesium using soybeans for his crop. He used a ratio of the calcium concentration divided by the sum of the calcium and magnesium concentrations to make evaluations. He determined that when the ratio of calcium to magnesium was small, depressed root growth occurred even when calcium was present in concentrations of 40 ppm. Taproot harvest length after 7½ days of growth was 299 mm with a calcium to magnesium ratio of 0.05. With a ratio of 0.20, the length was 485 mm.

Chloride

Chlorides are another of the essential nutrients for plant growth. Though generally not phytotoxic, in excessive amounts chlorides can cause leaf tip burn, premature yellowing, and at times chlorosis in some plants.

A number of investigators have worked in the area of chloride deficiency with interesting results. Raleigh (1948) studied the effect of chloride addition to a chloride deficient culture solution on table beets. He found that when several millequivalents of chloride were introduced, the yield increased significantly. Harward et al. (1956) working with Irish potatoes had similar results in increasing yields. Corbett and Gausman (1960) also studied potatoes with significant yield increases when chloride deficiencies were treated with added concentrations of chloride.

Table 5 (Eaton, 1966) shows some common plants and their suggested tolerance to excessive chloride concentrations. Note that fruit trees are relatively sensitive whereas certain vegetables and grains are rather tolerant. However, these limits can be readily modified by leaching. It is also important to realize that though these limits are helpful, usually salinity levels are more important than chloride levels because salinity will ordinarily reduce crop growth first.

A number of studies have investigated the toxic effects of excessive concentrations of chloride. Concentrations as low as 3 meq/l of chloride in irrigation water have caused injury to citrus, stone fruits, and almonds (Bernstein, 1967). Bingham et al. (1968) found that the application of 5 meq/l of chloride in irrigation water could constitute a hazard to avacado trees.

Foliar absorption of chloride has been found to be of importance when using sprinkler irrigation (Eaton and Harding, 1959; Ehlig and Bernstein, 1959). The difference in evaporation between day and night and the amount of evaporation occurring between revolutions of the sprinkler head can cause adverse effects.

Meron et al. (1965) in a study performed in Israel found that high chloride concentrations may cause slow but eventually fatal physiological injury to citrus trees. The authors also state that as of 1965, chloride concentrations in the range of 150 to 200 mg/l (4.15 to 5.55 meq/l, respectively) are accepted for irrigation of citrus trees, and chloride concentrations of 300 mg/l (8.3 meq/l) are accepted for other agricultural crops.

Table 5. Tolerances to chloride, as measured in sand and water cultures.[a]

Low Tolerance Class I		Medium Tolerance Class II		High Tolerance Class III	
Peach	13	Strawberry	20	Wheat (young)	25
Avocado	14	Apricot	20	Tomato	39
Lemon	15	Orange (Valencia)	20	Cotton	50
Prune	16	Rice	23	Flax	50
Bean (navy)	18	Sorghum	23	Corn (young)	70
Plum	18	Alfalfa	23	Barley	90
Dallis grass	19	Rhodes grass	24	Beet	100
		Bean (kidney)	24		

[a] Chloride concentrations in meq/l producing 80% growth or yield.

Source: Eaton (1966).

Copper

Evidence was presented by Sommer (1931) and by Lipman and MacKinney (1931) that copper was an essential nutrient for plant growth. Sommer (1931) found that when copper was eliminated from the diet of tomatoes, sunflowers, and flax that growth reductions up to 90 percent occurred. Lipman and MacKinney (1931) found that barley gave normal growth when fed one part cooper in ten million parts of nutrient culture solutions made from highly refined chemicals. When copper was omitted, the plant did not fruit properly. Because of a large amount of circumstantial evidence built up before these two papers, the evidence of 1931 was accepted.

In a study done on a copper deficient soil in Manitoba, Canada, Campbell and Gusta (1966) realized significant yield increases of carrots and onions with the application of only 0.5 pounds/acre of copper. An increase to 2.5 pounds/acre of copper gave no further increase but caused no adverse effects either. The quality of the onions was also improved.

Copper can act toxically if in excess quantities. Anne and Dupuis (1953) among others have shown that toxic amounts of copper can reduce growth and may depress iron concentrations in leaves causing iron chlorosis symptoms. Forbes (1917) showed that excessive copper could also cause root damage. Arnon (1950) found that a 2.0 mg/l concentration of copper in a nutrient solution was toxic to tomato plants. Smith and Specht (1953) found that a 0.1 mg/l concentration was toxic to orange and mandarin seedlings. Millikan (1949) found that a concentration of 0.5 mg/l in a water culture caused damage to flax. Hewitt (1953), using a sand culture, found that nutrient solutions containing 6.4 to 31.8 mg/l of copper caused damage to sugar beets, tomatoes, and potatoes but not to oats and kale. Tolerance levels have been suggested for copper in irrigation water which agree in general with the above findings. Values of 0.1 mg/l (McKee and Wolfe, 1963) and 0.2 mg/l (National Technical Advisory Committee on Water Criteria, 1968) have been suggested. Small discrepancies may be accounted for because irrigation water has an alkalyzing effect (increase of the exchangeable-sodium content of a soil) which may reduce copper availability and bring on deficiency. This is especially common in sands and high organic soils.

Hunter and Vergnano (1952) found that when oats received 2.0 ppm copper in a nutrient solution, they were usually normal, though some showed slight interveinal chlorosis. Those that received 10.0 ppm were very chlorotic. With concentrations of 20.0 ppm or greater, the plants were small and the leaves narrow. A few were chlorotic but most were orange colored. Above 2.0 ppm, roots showed a decrease in size.

Fluorine

Fluorine is not considered an essential element for plant growth. It can however, cause toxic effects in excess amounts. Injury appears as marginal necrosis (leaf tip burn) and/or interveinal chlorosis.

The amount of fluoride uptake by the plant has been shown conclusively to be independent of soil concentration by Hurd-Karrer (1950) among others. The determining factors seem to be soil type, calcium and phosphorus content, and pH. Hurd-Karrer (1950) also demonstrated that excess quantities of soluble fluorides applied to unlimed acid soils will cause plant injury. On the other hand, addition of soluble fluorides to well limed soils or the addition of insoluble fluorides to acid or limed soils caused no apparent injury. Soils at a pH of 6.5 will almost completely fix any soluble fluoride compounds in the soil or added to the soil.

A number of toxic effects caused by excessive fluorides have been noted. Leone et al. (1948) found that when 10 mg/l of fluorides were applied to tomato, peach, and buckwheat plants, no injury occurred. However, at levels of 100 mg/l, the plants were severely injured in three days. At levels of 200 mg/l, the plants were killed in a short time. In another study (Science News Letter, 1949) using beans, levels between 100 to 500 mg/l inhibited sprouting of the beans. At levels of 1000 mg/l the growth of the beans were markedly stunted. Prince et al. (1949) in other studies on buckwheat found that at levels of 180 mg/l no injury occurred at pH over 5.5. This result agrees with the results arrived at by Hurd-Karrer (1950). However, when the concentration was increased to 360 mg/l both peach and buckwheat were injured even at a pH of 6.5.

Brewer et al. (1959) used treatments of hydrogen fluoride at rates of 0, 25, and 100 ppm on four naval orange trees. All trees were 4 years old. All four trees receiving 100 ppm wilted within 24 hours after the application. Wilting was followed by complete defoliation. Root examination revealed a complete coverage of the roots by a slimy, gelatinous film. The film proved to be mainly calcium, phosphate, and fluoride ions. The trees were so badly damaged they were discarded. The other two treatments were continued for 18 months. During this time, the trees treated with 25 ppm became progressively less vigorous than the control trees. Spring leaf drop was also more severe. The foliage also became increasingly sparse. Tests run on the fruit showed the 25 ppm trees to be vastly inferior both in quantity and quality.

McNulty and Newman (1961) used concentrations of 1.31 x 10^{-2}M sodium fluoride and found a definite reduction in the amount of chlorophyll in bush beans. The carotenes of these plants were also reduced. Chang (1968) found that fluoride in concentrations of 5 x 10^{-4}M and 1 x 10^{-3}M suppressed the growth rate of corn seedling roots by about 20 percent and 40 percent respectively. Chang also pointed out that fluoride is known to act as an enzyme inhibitor.

Iron

Iron was demonstrated to be an essential nutrient over a century ago. Griss (as cited by Wallihan, 1966) found that foliar application of iron to chlorotic grapevines was very beneficial. Iron toxicity has not occurred to any great degree under natural conditions.

The most common symptom of iron deficiency is chlorosis which is a reduced concentration of chlorophyll in green plants. The addition of inorganic iron salts generally produces little or no effect because such compounds are quickly rendered insoluble in alkaline soils. This would also explain why problems of toxicity have not occurred under natural conditions.

Lead

Lead is not considered an essential plant nutrient although there is evidence of some small beneficial effects. Toxicity results for lead have been somewhat contradictory. Klintworth (1952) stated that lead is harmful to plants at all concentrations. On the other hand, Frear (no date) added lead nitrate to oats and potatoes at concentrations of 1.5 to 25.0 mg/l and got stimulation effects. When he increased the dosage to 50.0 mg/l, all plants died within a week. Hewitt (1953) found that when sugar beets were grown in a sand culture, the beets showed a slight injury with a nutrient solution containing 51.8 mg/l of lead.

Hooper (1937) found that when dwarf French beans were grown in a nutrient culture containing 20 ppm lead in the form of lead sulphate, the plants suffered no apparent damage. In a spraying experiment, lead spray again was found not to harm the plants other than by spraying heavily enough to physically block the stomates.

Magnesium

Magnesium was found to be an essential nutrient around the turn of the century by Pfeffer (as cited by Embleton, 1966). A few years later, Meyer and Anderson (1939) affirmed this conclusion. Studies during this period also indicated possible toxicity problems associated with excess magnesium.

Reports of magnesium toxicity seem to indicate the problem is one of imbalance with one or more other elements. These problems could perhaps be overcome by increasing the level of the particular element or elements without increasing the magnesium level (Embleton, 1966). This presumes that by increasing the other element levels, the problem is not merely shifted but rather solved. Each case would require individual analysis.

Garner et al. (1930) showed that soils low in calcium which are given applications of magnesium may produce toxic effects. Kelley (1948) found that when a soil had more than 90 percent of the cation exchange capacity saturated with magnesium, the soil was almost totally unproductive. Gauch and Wadleigh (1944) found that concentrations of 3000 to 5000 mg/l of $MgCl_2$ and $MgSO_4$ were toxic to bean plants.

Magnesium and calcium ions in irrigation water tend to keep soil permeable and in good tilth. (See Water for Irrigation Use, 1951.) This is expected because the SAR is inversely proportional to the concentrations of these two ions. More will be said in the section on sodium concerning this area.

Manganese

Many early researchers have shown the necessity of manganese as an essential plant nutrient. Bertrand (as cited by Labanauskas, 1966) was one of the first to advance the idea. McHargue (1926) grew wheat with and without manganese and found that without manganese the wheat became chlorotic, was stunted, and produced no seed. These effects were absent in the wheat grown with manganese.

Further research showed that manganese in excess quantities could cause toxic effects. Deatrick (1919) showed that small concentrations of manganese clearly stimulated wheat growth but as the concentration became large, detrimental effects appeared. Bishop (1928) working with radishes, beans, corn, and peas found that concentrations of 5 ppm of applied manganese stimulated growth whereas concentrations of 10 ppm or greater depressed growth. He found radishes, however, to be better at 10 ppm and declined from that point provided the pH was neutral rather than acidic.

Joham and Amin (1967) determined that 81 ppm of applied manganese was toxic to cotton. The optimum level was found to be on the order of 3 ppm. Good growth and fruiting were obtained, however, up to levels of 27 ppm.

It seems that certain soil properties increase the probability of excess manganese problems. Snider (1943) has shown that soils that are strongly acidic frequently have excess manganese problems. This is because manganese is rather dependent on pH in its availability to plants. In other studies, Fergus (1954) showed, using French beans and peanuts, that the amount of manganese uptake by the plants was pH dependent. He concluded that where large amounts of manganese were in the soil, at pH values greater than 5, good growth could be expected. However, at pH values less than 5, symptoms of toxicity would probably appear. Table 6 is a portion of the data taken during this study. The proof of the above statements is rather obvious from the table.

Results in full agreement with Fergus' (1954) work were published by Gupta et al. (1970). They found that when manganese was applied to carrots at concentrations of 1000 ppm in a soil with a pH of 5.7 or higher beneficial results occurred. These included reduced bronzing of leaves and increased yields. Brown et al. (1968) showed that sugar beets were very tolerant to manganese. In one experiment, pots of sugar beets were treated with manganese sulfate at the rate of 650 ppm manganese with no apparent toxic effects. As the above studies would suggest, the pH was approximately 5.5.

Table 6. Plant growth in relation to soil pH and manganese content of plant.

Soil pH	Plant Mn (ppm)	Plant Growth
4.4	3000	Very poor
4.5	2400	Poor
4.8	1200	Fair. Some chlorosis
4.9	800	Good
5.0	1100	Fair. Some chlorosis
5.9	260	Good

Nitrogen

Nitrogen is an essential element for plant nutrition and the one most commonly associated with plant health. To a large extent it controls the growth and fruiting of most plants. One of the early milestone pieces of work establishing the need by plants for nitrogen was performed by Kraus and Kraybill (1918). They found that by using nitrates tomato plants with sufficient nitrogen nutrition did well whereas those without nitrogen did not do well at all. Nitrogen shortages are more likely to be a problem than nitrogen excesses because of the high mobility of the element and its compounds. Shortages result in a reduction of chlorophyll resulting in a loss of green color.

Nitrogen excesses can cause several kinds of problems. Embleton et al. (1959) using sulfate of ammonia for 3 pounds nitrogen found that plants become highly vegetative resulting in yield reductions. Clements (1958) found that excess nitrogen reduced sugar production in sugar cane. In other studies (Jones and Embleton, 1959; Reuther et al., 1958) using urea and different nitrates, the quality of different fruits was found to be impaired. In excess, nitrogen can also add to the salinity of the soil. This has been confirmed by Pratt et al. (1959). Their nitrogen sources included urea, manure, dried blood, various nitrates, and ammonium sulfate. Excess nitrates tend to reduce soil permeability. Nitrates may accumulate to toxic levels but their effect is usually osmotic (Thorne and Peterson, 1949).

All soils will become deficient in nitrogen in time if there is no replenishing of the nitrogen using some type of fertilization program. Soils especially subject to deficiency, however, are sandy soils subject to high rainfall causing leaching and soils low in organic matter.

Zanoni et al. (1967) performed studies showing that if carbohydrate reserves are depleted to low levels in response to higher soil temperatures and these levels are further reduced by increased nitrogen levels, grasses appear to be more susceptible to injury from disease. This happens because at temperatures greater than 60°F, top-growth is stimulated thus drawing on the root reserves. Therefore excess nitrogen during warm weather can indirectly cause injury by increasing greatly the disease potential.

Ford et al. (1957) using NH_4NO_3 showed that over fertilization with nitrogen can reduce the concentration of feeder roots in the principal rooting zone of citrus trees on sandy soil. Neither the time of application nor the fertilizer ratio was a factor in the shift in root concentration associated with high nitrogen. Treatments were initially 0.6, 1.2, and 2.4 pounds of nitrogen per tree per year. This was gradually increased to 0.87, 1.75, and 3.5 pounds as the trees grew.

Phosphorus

Phosphorus was established as an essential nutrient in 1839 and 1840 by Liebig in Germany and Laws in England, respectively. (See Bingham, 1966.) Along with nitrogen and potassium, phosphorus is quite commonly supplied to the plant by application of commercial fertilizer. Phosphorus ties up in the soil readily and therefore only a small fraction of the phosphorus in the soil is available for plant use. Acidification of the soil will help to make phosphorus more available to the plant.

The soils most likely deficient in phosphorus are highly weathered soils, calcareous soils, and peat and muck soils. Highly weathered soils are usually acidic and often deficient in available phosphorus. Calcareous soils may have a large total phosphorus but the high pH due to the calcium makes the phosphorus mostly unavailable. Peat and muck soils often respond well to phosphorus fertilization.

Cannell, et al. (1960) performed studies on tomato plants and found that phosphorus applications of 2, 10, 26, and 50 grams of $Ca(H_2PO_4)_2{\cdot}H_2O$ per 12.5 kg of soil tended to decrease the adverse effect of soil suction. This resulted in larger and greener plants. These experiments were conducted on three soil types, acidic, moderately acidic, and alkaline. The authors suggest that perhaps phosphorus in the form used lowered the pH and made certain ions more available to the plants, i.e., became more soluble. They cited manganese and magnesium as examples.

A number of problems resulting from phosphorus toxicity studies have been noted. Rossiter (1952) found that the nodulation of legumes was reduced. Bingham and Garber (1960) have shown a phosphorus-copper antagonism, and Loneragan (1950) presented evidence for a phosphorus-zinc antagonism. Phosphorus-iron antagonisms were discussed by Brown et al. (1968). In many cases, the bad effects of phosphorus in view of the antagonisms mentioned above would appear to be indirect. These interactions are generally more readily produced in acid soils.

Bingham (1966) found that with a water soluble concentration of 20 to 100 ppm or more of phosphorus, copper deficiency was induced in citrus seedlings by high phosphate additions. He suggests that 40 to 80 pounds of P_2O_5 per acre be used on small grains and 60 to 120 pounds of P_2O_5 per acre on vegetables. Alfalfa requires high rates, from 160 to 320 pounds of P_2O_5 per acre. Where needed, citrus should initially receive 4 to 5 pounds of P_2O_5 per tree and a maintenance amount of 1 to 2 pounds per tree every few years.

Generally when phosphates occur in irrigation waters, they offer beneficial fertilizing effects (Joshi, 1945). Blueberry plants grown in nutrient water of low phosphate content (1 to 5 mg/l) showed symptoms of phosphate deficiency. When the concentration was increased to 60 mg/l, signs of incipient iron chlorosis appeared. Phosphorus seems to reduce the amount of available inorganic iron thereby producing the observed chlorosis (Holmes, 1960).

Potassium

Potassium was established as an essential nutrient over one hundred years ago by Birner and Lucanus (Ulrich and Ohki, 1966). Using oats, they found that potassium was essential to flowering and could not be replaced by any metal of the same grouping. Potassium problems generally occur as deficiencies rather than as excesses. The reason is that both exchangeable and non-exchangeable potassium are fixed in the soils. There are some indications that potassium may cause deficiencies of other nutrients. Reuther and Smith (1954) for instance suggested that the absorption of zinc and iron by some plants may be adversely affected by excesses of potassium. Potassium deficiencies will manifest themselves as interveinal chlorosis near the margins progressing to what is termed leaf scorch. Table 7 (Ulrich and Ohki, 1966) shows several crops with values of potassium found to be minimum levels. These values were established by finding the point at which potassium fertilization no longer provided significant response.

Potassium, while essential for plant growth, must be maintained in proper balance to other nutrients such as phosphorus (Wilcox, 1948). In another study (Water for Irrigation Use, 1951), it was found that potassium in irrigation water can act on soils somewhat like sodium only it is less harmful.

Table 7. Some exchangeable potassium levels below which responses to potassium fertilization have been reported (Ulrich and Ohki, 1966).

Plant	Pounds of Exchangeable Potassium (K) Per Acre of Soil[a]
Alfalfa	80
	160
	180
Corn	155
	83
	200
	150
Cotton	185
	120
Field crops	200
Grape	200
	136
Pineapple	156
Potato	140-375
	220
Sugar cane	150
	180
	200
Tobacco	190

[a] Basis: 2,000,000 pounds of soil per acre.

Salinity

Increased salinity causes problems by increasing the osmotic pressure of the soil for water. As salinity increases, the plant has a more difficult time obtaining water from the soil. The relationship expressing total soil suction on a plant is

$$TSS = MS + SS$$

where TSS is the total soil suction withholding water from the plant, MS is the matric suction or the physical attraction of soil particles for water, and SS is the osmotic pressure due to salinity. As water is lost through evapotranspiration, the salt concentration builds up rapidly causing the term SS to become increasingly important. The matric suction increases exponentially as water is lost. The combined effect of the two factors can cause critical conditions for soil water availability. (See Table 8 for general guidelines for salinity in irrigation water.)

Plants react differently to salinity problems depending on the growth stage at which salinity becomes a problem. A study (Lunin, 1963) was conducted showing the response of beets to salinity at the seedling stage and at 2 and 4 weeks of maturity. Both the roots and the top growth were studied. The results indicated tops were affected about the same. Roots, however, were highly dependent on the stage of plant growth.

Climatic factors such as wind and temperature are important in determining evapotranspiration rates which relate back to osmotic pressure. Magistad et al. (1943) demonstrated the effects of climate on salinity levels. They found using cotton, alfalfa, sugar beets, tomatoes, squash, onions, navy beans, garden beets, and carrots that at a given salt concentration these crops are depressed in relative yield more in warm than in cool climates. In humid regions, irrigation water is used to supplement rainfall. Therefore a given water will be less likely to cause salinity problems. This is true because salt accumulation is a function of the salt concentration of the irrigation water, evapotranspiration, drainage and leaching, and the amounts of water applied and received as precipitation.

Leaching is the best method of controlling the salinity problem. Adequate leaching depends on a number of factors. First, the drainage must be sufficient to wash the salts below the root zone. Otherwise root systems will be subjected to the problems of osmotic pressure and not take in sufficient water. Another factor of equal importance is the proper amount of water for leaching. Not enough will leave the salinity problem only partially solved while too much will clog the pore spaces of the soil and displace the oxygen in the soil. When this condition exists for a prolonged period of time, plant roots may actually rot away killing the plant. During a field trip a row of trees was pointed out by the field trip leader, which due to excess wetness of the root zone, had nearly died. The problem was determined and the appropriate action executed preventing the loss of the trees.

Saline waters must be used in greater amounts than non-saline waters because more of the saline water must pass beyond the root zone than that of the non-saline water. When saline water is being considered for irrigation, the individual making the decision should only consider well drained soils.

In a study performed by Robinson et al. (1968), sprinklers were shown to be more effective in the removal of soil salts. The water used was from the Colorado River with a salinity of approximately 12 meq/l. They found that it was necessary to sprinkle more frequently than usual to avoid leaf tip burn due to salt accumulation. They used clay and sandy clay loam soils and grew sugar beets, cabbage, carrots, onions, wheat, barley, safflower, and flax. Under these conditions they were able to successfully grow these crops. Peterson (1968) warns against sprinkler use however. He points out that water applied by sprinklers may cause problems which would not occur using surface application. Salt buildups can occur causing tip burn, marginal leaf burn, or even defoliation of sensitive plants.

Bernstein (1965) pointed out that most fruit crops are more sensitive to salinity than are field, forage, or vegetable crops. He also pointed out the difference in sensitivity of the plant to salinity at different stages of growth. This is in agreement with Lunin et al. (1963).

Table 8. General guidelines for salinity in irrigation water.

Classification	TDS (mg/l)	EC (mmhos/cm)
Water for which no detrimental effects are usually noticed	500	0.75
Water that can have detrimental effects on sensitive crops	500-1,000	0.75-1.50
Water that can have adverse effects on many crops, requiring careful management practices	1,000-2,000	1.50-3.00
Water that can be used for tolerant plants on permeable soils with careful management practices	2,000-5,000	3.00-7.50

Source: National Academy of Science--National Academy of Engineering, Environmental Study Board, ad hoc Committee on Water Quality Criteria 1972: Water Quality Criteria 1972, U.S. Government Printing Office (in press).

Werkhoven et al. (1966) found that deciduous trees were more salt tolerant than the coniferous species. In terms of survival, an EC_e value of between 7 and 10 mmhos appeared to be critical for caragana and elm in a soil with a moisture content of about midway between the wilting point and field capacity. The corresponding values for spruce and pine were closer to 4 and 6 mmhos respectively. Seedling survival was markedly improved by maintaining the soil moisture level at field capacity. Emergence seemed to improve with some salinity, but survival was definitely adversely affected by the salinity.

Table 9 (Eaton, 1939) gives suggested salt tolerances for a number of crops. The table is based on the criteria of the relative yield of the crop on a saline soil as compared with its yield on a non-saline soil under similar growing conditions. The table displays the crops in columns with the most tolerant crop at the top and the least tolerant crop at the bottom. The values of EC_e given represent the levels of salinity at which a 50 percent decrease in yield would be expected compared to non-saline soils under similar growing conditions. Of course climatic conditions may greatly influence plant reactions compared to these suggested limits.

Table 9. Relative tolerance of crop plants to salt, listed in decreasing order of tolerance (Eaton, 1939).[a]

High Salt Tolerance	Vegetable Crops Medium Salt Tolerance	Low Salt Tolerance
$EC_e X10^3=12$	$EC_e X10^3=10$	$EC_e X10^3=4$
Garden beets	Tomato	Radish
Kale	Broccoli	Celery
Asparagus	Cabbage	Green beans
Spinach	Bell pepper	
	Cauliflower	
	Lettuce	
	Sweet corn	
	Potatoes (White Rose)	
	Carrot	
	Onion	
	Peas	
	Squash	
	Cucumber	
$EC_e X10^3=10$	$EC_e X10^3=4$	$EC_e X10^3=3$
High Salt Tolerance	**Field Crops Medium Salt Tolerance**	**Low Salt Tolerance**
$EC_e X10^3=16$	$EC_e X10^3=10$	$EC_e X10^3=4$
Barley (grain)	Rye (grain)	Field beans
Sugar beet	Wheat (grain)	
Rape	Oats (grain)	
Cotton	Rice	
	Sorghum (grain)	
	Corn (field)	
	Flax	
	Sunflower	
	Castorbeans	
$EC_e X10^3=10$	$EC_e X10^3=6$	
High Salt Tolerance	**Forage Crops Medium Salt Tolerance**	**Low Salt Tolerance**
$EC_e X10^3=18$	$EC_e X10^3=12$	$EC_e X10^3=4$
Alkali sacaton	White sweetclover	White Dutch clover
Saltgrass	Yellow sweetclover	Meadow foxtail
Nuttall alkaligrass	Perennial ryegrass	Alsike clover
Bermuda grass	Mountain brome	Red clover
Rhodes grass	Strawberry clover	Ladino clover
Rescue grass	Dallis grass	Burnet
Canada wildrye	Sudan grass	
Western wheatgrass	Hubam clover	
Barley (hay)	Alfalfa (California common)	
Bridsfoot trefoil	Tall fescue	
	Rye (hay)	
	Wheat (hay)	
	Oats (hay)	
	Orchardgrass	
	Blue grama	
	Meadow fescue	
	Reed canary	
	Big trefoil	
	Smooth brome	
	Tall meadow oatgrass	
	Cicer milkvetch	
	Sourclover	
	Sickle milkvetch	
$EC_e X10^3=12$	$EC_e X10^3=4$	$EC_e X10^3=2$
High Salt Tolerance	**Fruit Crops Medium Salt Tolerance**	**Low Salt Tolerance**
Date palm	Pomegranate	Pear
	Fig	Apple
	Olive	Orange
	Grape	Grapefruit
	Cantaloupe	Peach
		Lemon

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[a] The numbers following $EC_e X10^3$ are the electrical conductivity values of the saturation extract in millimhos per centimeter at 25°C associated with 50 percent decrease in yield.

Selenium

Though some studies have shown a stimulation of plant growth from applications of very low concentrations of selenium (Trelease and Trelease, 1938), generally selenium is not considered beneficial to plants. In fact, problems associated with selenium are normally those of toxicity due to excess selenium.

The literature indicates that the primary problem of selenium is with forage crops because of the potential poisoning of grazing animals. Trelease and Beath (1949) have prepared a comprehensive discussion of specific symptoms of selenium poisoning in animals.

In the continental United States, selenium producing seleniferous plants have been found only in arid or semi-arid areas with mean annual rainfall less than 20 inches (Trelease and Beath, 1949). Hough et al. (1941) found 12 to 14 ppm of selenium in some lava produced soils in Hawaii. This is the highest selenium content encountered in soils which do not produce a seleniferous vegetation.

Bisbjerg and Gissel-Nielsen (1969) tested a variety of plants and found them to be injured significantly by applications of 2.5 ppm or higher. The plants included in their study were clover, lucerne, radish, black medick, white mustard, perennial rye grass, rye, wheat, barley, and oats. Hurd-Karrer (1935) stated that selenium injury to plants can be reduced or prevented when the concentration of sulfate ion is about twelve times greater than the selenium concentration. Table 10 (Miller, 1954) suggests tentative limits of selenium in irrigation water.

Table 10. Tentative limits of selenium in water for irrigation (Miller, 1954).

Irrigation Class	Selenium (mg/l)	Remarks
1. Low	0.00-0.10	No plant toxicity anticipated
2. Medium	0.11-0.20	Usable, but with possible long-term accumulations under particular conditions should be watched
3. High	0.21-0.50	Doubtful-probable toxic accumulation in plants except under especially favorable conditions
4. Very high	over 0.50	Non-usable under any conditions

Silica

Up until recently silica was generally considered of little importance in irrigation water (Water for Irrigation Use, 1951). The U.S. Department of Agriculture has recommended limits of 10 to 50 mg/l (McKee and Wolfe, 1963).

Later studies have increased the importance of silica however. Eaton et al. (1968) found that magnesium in the form of silicates can be lost from the soil by precipitation under the following conditions: (1) The concentration of magnesium must be on the order of several meq/l, and (2) there must be a degree of alkalinity such that a part of the silicon dioxide is ionized as silicates. They concluded that precipitations of calcium carbonate by whatever mechanism occurred and the associated increases in sodium percentages provided the source of alkalinity. Thus silica may indirectly affect the SAR of the soil system.

Silver

Silver has not been shown to be an essential element in the nutrition of plants. Problems of toxicity are unlikely because of the relative insolubility of silver and its common compounds. The element availability to plants is thus low. Vanselow (1966) found that citrus leaves growing in soil to which 75 ppm of silver as nitrate had been added contained only 0.5 ppm. This does not seem to cause any injury to the plant at all.

Sodium

Except for two or three plants, sodium is not an essential nutrient for higher plants (the poisonous weed Halogeton is one exception). However, beneficial results have been reported. Cope et al. (1953) have found that in some soils, sodium applications have increased the available potassium supply. Truog et al. (1953) found that sodium applications to celery improved the taste and crispness markedly. In the same study, carrots exhibited improved taste (sweeter) with sodium applications.

Sodium can also produce detrimental effects on plants and in the soil. The sodium condition in soils can be evaluated using the sodium-adsorption-ratio which is expressed as

$$\mathrm{SAR} = \frac{\mathrm{Na}^{+}}{\left(\frac{\mathrm{Ca}^{++} + \mathrm{Mg}^{++}}{2}\right)^{1/2}}$$

where the ions are concentrations expressed as milliequivalents per liter. Another means of defining the sodium condition is the exchangeable-sodium-percentage, abbreviated ESP. It is related to the SAR by the following equation:

$$\mathrm{ESP} = \frac{100(-0.0126 + 0.01475\ \mathrm{SAR})}{1 + (-0.0126 + 0.01475\ \mathrm{SAR})}$$

When 10 to 20 percent of the cation-exchange-capacity of a soil is taken up by sodium, the soil condition exhibits a significant deterioration (Martin and Richards, 1959). The soil becomes very hard physically. This condition is sometimes referred to as alkali soil or as chemical compaction.

Sodium, in addition to affecting plants by damaging the soil, can also affect plants directly. Harding et al. (1958) demonstrated that certain plants can absorb sodium through their leaves if irrigated using sprinkler irrigation. Ehlig and Bernstein (1959) showed that leaves can absorb sodium rapidly enough to cause injury to the leaves. Since citrus trees accumulate sodium readily through their foliage, these plants are rather sensitive to sodium injury. In fact sufficient sodium has been absorbed by citrus leaves from a single sprinkling of water containing 69 to 100 mg/l of sodium to cause serious leaf burn and defoliation (Fireman, 1958).

Sodium sensitive plants such as woody plants may accumulate harmful levels of sodium in the leaves when the ESP of the soil is about 5 percent. The SAR tolerance limit for water used on fruit crops is about 4. For nonsensitive crops where soil effects predominate, the SAR value can be from 8 to 18 (Bernstein, 1967). Bernstein and Pearson (1956) found that an ESP of 15 can cause nutritional disturbances to green beans, beets, clover, and alfalfa. Soils that are permeable often can be easily leached. However, soils in an advanced state of sodium excess are less permeable and consequently more difficult to handle.

One exception to the above ESP values was described by Werkhoven et al. (1966). They found that safflower showed greatly increased growth as measured by dry weight of tops when the ESP was increased to 20 and 30 percent. ESP levels greater than 30 percent were detrimental to both growth and yield.

Sulfate

Sulfur has been known to be an essential nutrient for over 100 years. Plants get sulfur from the sulfates in the soils. Problems with sulfates usually are more frequently associated with deficiencies rather than excesses. Toxic effects however have been noted.

Many areas in the world are experiencing severe sulfur shortages. Coleman (1966) points out the reasons for these shortages are: (1) the increasing use of sulfur-free fertilizers, (2) the decreasing use of sulfur as a pesticide, and (3) increasing crop yields which require greater amounts of plant nutrients. He also mentions the ease of sulfate leaching which contributes to the shortage.

Deficiencies of sulfur appear visually much like that given by nitrogen deficiency (Ergle, 1953). However, Gilbert (1951) found that with plants in general, the older leaves did not dry up as did those with nitrogen deficiency. Eaton (1942) using a sand culture with up to 250 meq/l of substrate sulfate found reductions in growth of several plants. Leaf damage also was noted in sorghum and navy beans. Figure 6 (Eaton, 1966) shows more clearly the results. Note that concentrations of sulfate greater than about 5 meq/l begin to cause detrimental effects. The rate of growth reduction does seem to slow down at 100 meq/l although damage is still continuing. Scofield (1935) suggested tolerance limits for sulfates in irrigation waters which are listed in Table 11. These values generally agree with Eaton's (1942) work. Hinman (1938) however, was of the opinion that concentrations in excess of 500 mg/l were generally hazardous. This tolerance limit is somewhat more limiting, of course, but not greatly so.

For further information regarding sulfur nutritional requirements, see Beaton (1966). He presents a review of sulfur needs for a rather large variety of plants.

Table 11. Tolerance limits for sulfates in irrigation water.

Rating	Tolerance
Excellent	Less than 192 mg/l
Good	192 to 336 mg/l
Permissible	336 to 576 mg/l
Doubtful	576 to 960 mg/l
Unsuitable	Greater than 960 mg/l

Zinc

Zinc was established in the early 1930's as an essential nutrient for plant growth. It was widely accepted as an essential nutrient after the work of Sommer and Lipman (1926) and Sommer (1928). In the former study, sunflowers and barley were shown to suffer significant injury in the absence of zinc. The later work increased the number of plants under study including buckwheat, Windsor beans, and red kidney beans for which zinc was demonstrated to be essential.

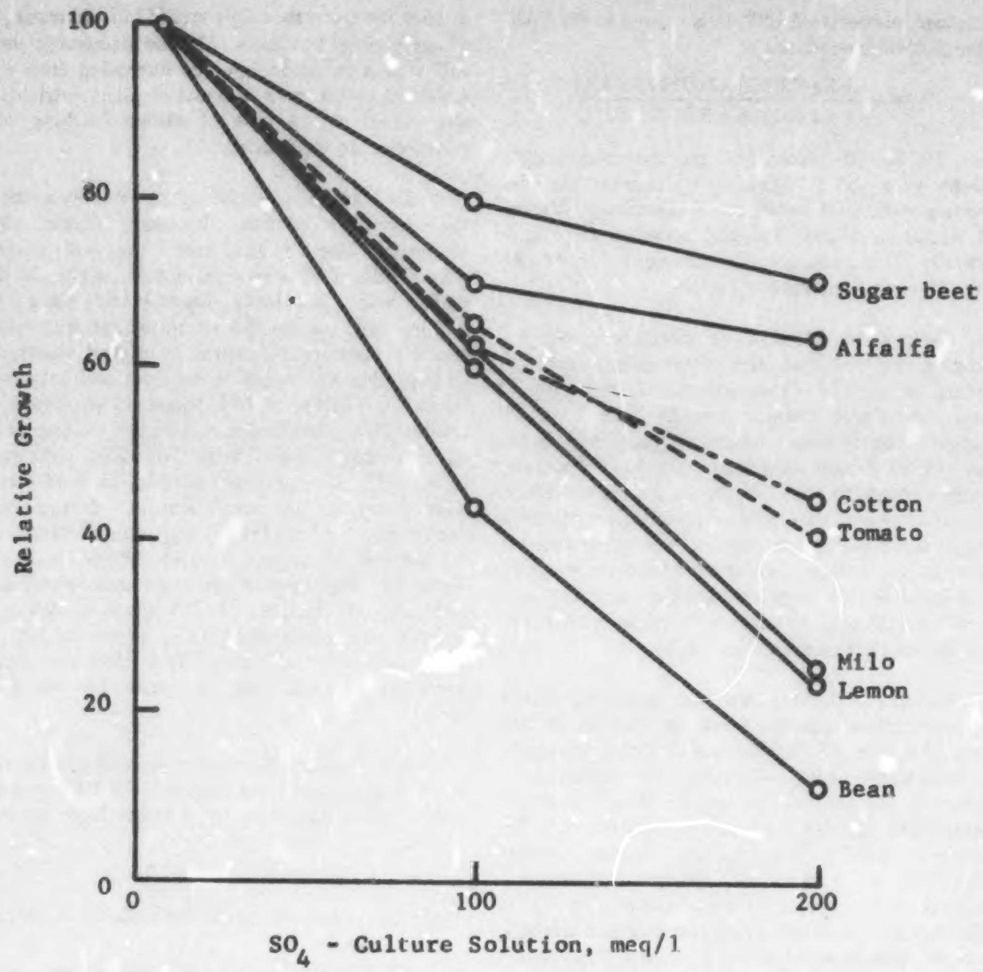

Figure 6. Growth depression of seven crops as influenced by varying sulfate concentrations.

When zinc is deficient, the plant leaves become mottled. A rosette-type terminal growth also can occur. The appearance is an absence of leaves on a branch except at the end of the branch where several leaves appear in a spray arrangement.

Zinc can also cause toxic effects. Excess zinc commonly causes iron chlorosis. Excesses occur in some kinds of acid peats and especially around mining operations where seepage and dumps can pollute the soil. In one study (McKee and Wolfe, 1963), certain concentrations of zinc in nutrient solutions were found to be toxic to plants. These are shown in Table 12.

Zinc deficiencies usually occur in soils that are acid, leached, and sandy; in alkaline soils because of decreased availability of zinc; organic soils where the zinc is tied up in forms not available to plants; and soils containing clays with low silica to magnesium ratios which give rise to forms of zinc not readily available to plants (Elgabaly, 1950).

Hunter and Vergnano (1952) found that with 10 ppm zinc in a nutrient solution, oats were normal. At 25 ppm the plants became slightly chlorotic. At

Table 12. Some toxic levels of zinc.

Zinc Conc. (mg/l)	Plant
3	Orange and mandarin seedlings
≥ 5	Flax
≥10	Water hyacinths
25-100	Oats

concentrations of 100 and 150 ppm, the plants were stunted and very chlorotic, and many leaf tips were yellow-red. Roots were normal at concentrations of 75 ppm or less, but were small at concentrations of 100 and 150 ppm.

Public health aspects

In California in 1918, regulations were instituted prohibiting the use of raw sewage, septic or Imhoff tank effluents, or water polluted by such sewages for the irrigation of most garden truck crops usually eaten raw by humans. For those crops that were cooked first, these waters were permissible provided irrigation with polluted waters was stopped 30 days prior to harvest. Exceptions to this rule were made, such as fruit and nut trees and melons, provided none of the fruit or vines made physical contact with the sewage. Later, in 1933, these rules were liberalized somewhat provided adequate disinfection procedures were used. This allowed the use of these effluents on garden truck to be eaten raw. However, the effluent quality after treatment had to be approximately that of drinking water (Ward and Ongerth, 1970).

In 1967, the Water Quality Control Act was revised. It generally differentiated water reuse and allowed standards to be set based on the intended use. Table 13 (Ward and Ongerth, 1970) shows the quality standards instituted for irrigation. As can be seen from the table, the risk involved to humans correlates closely to the severity of the requirement. The standard for food eaten raw is that the effluent be free of enteric viruses, at least those possible to detect by existing methods and that the hepatitis agent not be present.

Considerable work on this problem has also been done in Israel (Meron et al., 1965). In 1954, the Israel Ministry of Health published interim regulations for certain restricted crops. These regulations have been revised from time to time but include the following "allowable" uses: (1) industrial crops unfit for human food, such as sugar beets, cotton, and fibres; (2) pasture, provided the animals shall not be allowed on the field until the grass is entirely dry; (3) grass grown for "dry" hay; (4) vegetables which are consumed only after cooking, i.e., eggplant, potatoes, sweet potatoes, maize, and dry onions; (5) the following trees: citrus, banana, nut, date palms, avocados, and all sapling nursery stock; (6) ornamental shrubs, plants, and flowers; (7) plants grown for seed production only; (8) sunflowers and carobs (St. John's bread), provided irrigation is in furrows only; and (9) apple, pear, and plum trees, provided irrigation is stopped at least one month before harvesting.

Detention in any oxidation pond for 5 days under aerobic conditions is accepted as minimum treatment to qualify a water for the irrigation of restricted crops. Effluent from standard secondary treatment facilities is similarly accepted for irrigation of these crops. After 20 days detention time in an aerobic pond, the effluent, with chlorination, may be used for any crop. These regulations reflect the water and land scarcity in Israel. The health authorities recognize they are taking calculated risks and so they

Table 13. Summary of statewide standards for the safe direct use of reclaimed wastewater for irrigation and recreational impoundments for California (Ward and Ongerth, 1970).

Use	Description of Minimum Required Wastewater Characteristics			
	Primary[a]	Secondary and Disinfected	Secondary Coagulated, Filtered[b] and Disinfected	Coliform MPN/100 ml Median (Daily Sampling)
Irrigation				
Fodder crops	X			No requirement
Fiber crops	X			No requirement
Seed crops	X			No requirement
Produce eaten raw, surface irrigated		X		2.2
Produce eaten raw, spray irrigated			X	2.2
Processed produce, surface irrigated	X			No requirement
Processed produce, spray irrigated		X		23
Landscapes, parks, etc.		X		23
Creation of impoundments				
Lakes (aesthetic enjoyment only)		X		23
Restricted recreational lakes		X		2.2
Nonrestricted recreational lakes			X	2.2

[a] Effluent not containing more than 0.1 ml liter hr.

[b] Effluent not containing more than 10 turbidity units.

maintain careful observation of public health conditions.

In 1953, a study on the bacteriological aspects of using polluted water on vegetables was conducted by Norman and Kabler (1953). They showed that coliform counts on vegetables are a consequence of the soil count and therefore of the irrigation water. Coliform counts of leafy vegetables such as lettuce, celery, and cabbage tend to be greater than smooth skinned vegetables such as tomatoes and peppers. In one test, the ratio between leafy to smooth skin was 40 to 1 using an irrigation water with the same media coliform count for both vegetable types. *Salmonella* was found in 11 of 16 irrigation water samples but in only 1 of 7 soil samples and in none of 10 vegetable samples. The authors pointed out that the length of time between the last irrigation application and the time of sample collection may be the reason for the low *Salmonella* counts.

In an earlier study, Rudolfs et al. (1951) investigated the problem using tomatoes. They showed that contamination was independent of height of the tomato above the ground. They also found the contamination to be independent of splashing from soil due to rain. The study showed that sunshine significantly lowers coliform counts. They found that cracked tomatoes had higher coliform counts than the uncracked ones. This was not surprising in view of the work of Norman and Kabler (1953) with leafy vegetables. One finding of the study was particularly interesting. When coliform suspensions were sprayed directly on the fruit, by the end of 35 days after spraying, the fruit had levels no higher than the control fruit. It was also shown that *Salmonella* levels were down to nearly nil at the end of 6 days as also were *Shigella*. Based on these findings, the authors concluded that if sewage irrigation is stopped one month before harvest, the fruit, if eaten raw, would not be likely to transmit human bacterial enteric diseases. They also concluded that except for tomatoes with abnormal stem ends (cracks, etc.), there was no material difference in coliform counts per gram of tomato on three differently irrigated plots; one plot was irrigated with settled sewage concurrently with growth, one plot was irrigated before planting only, and one was not irrigated with sewage at all.

Up to this point one might think the problems are minor. That in fact is not the case. Wright (1950) found that there is evidence indicating the consumption of fresh vegetables irrigated with sewage or polluted water has caused many disease outbreaks. Epidemics of typhoid fever and parasitic infestations traceable to this cause have occurred in many areas of the world including the United States. Certainly the strict precautions observed in California seem justified until more research can be done to better understand the problem and to develop means to control these problems.

Soil clogging

The phenomena of clogging is important to soil and crop systems. Clogging resulting from the application of sewage effluent can reduce the supply of moisture, air, and nutrients to plants to a point where severe crop damage can occur.

Thomas et al. (1966) used a sewage effluent from a septic tank with a BOD_5 of 87 mg/l to treat a sand soil. The effluent was applied daily at a rate of 5 gal/day/ft^2. The sewage was applied as nearly as possible instantaneously and then the bed allowed to drain until the next application. Clogging occurred in three phases. The first phase was a gradual reduction in filtration rate; the second phase was a short period characterized by a rapid decline in infiltration rate; in the third phase, the infiltration rate asymptotically approached a lower limit. Clogging was shown to be a surface phenomena that occurred in the top 1 cm principally, but did occur on down to 6 cm. Total organic matter was shown to be a significant contributor to clogging. The authors also suggested the possibility that iron and phosphate may contribute to the problem.

Jones and Taylor (1965) also found that clogging occurred in three phases. Their phase descriptions differ somewhat from Thomas et al. (1966). Jones and Taylor (1965) describe the three phases as follows: The first phase is a period in which the conductivity declines to nearly 25 percent of its initial value; during the second phase the hydraulic conductivity declines slowly to nearly 10 percent of the original value; a rather sharp drop constitutes the third phase, and the conductivity drops to 1 or 2 percent of the initial value. Under anaerobic conditions, the second phase of clogging is absent and the first and third phases are indistinguishable. The reason for the discrepancy in the descriptions of the second phase is not clear, but perhaps has to do with different soil types.

In this same study, Jones and Taylor (1965) found that soil clogging occurs three to 10 times faster under an anaerobic environment than under an aerobic one. They also found that sands of high initial hydraulic conductivity are clogged at a much slower rate than ones of low initial conductivity.

Rather surprisingly, Avnimelech and Nevo (1964) claimed that organic materials which decompose slowly do not cause biological clogging of sands. Included in the list of organic materials that would not induce clogging because of slow decomposition was sewage sludge. This study seems inconsistent with the well known need for periodic resting of septic tanks. The need for scraping the tops off sand filters would also seem inconsistent with this study.

Discussion

Introduction

This section consists of a summary of the discussion of water quality factors in relation to specific crops. A list of crops that appear suitable for irrigation by each effluent is also included. The list includes only those plants specifically mentioned in this paper. Following this is a section of case histories to show successful utilization of sewage effluent for irrigation.

Table 14 summarizes the results of the study of chemical toxicities. These values represent the most sensitive levels reported for each component. Specific cases may be less sensitive depending on climate, soil type, etc. These values are approximately equivalent to levels accumulated in nature over an extended period of time. The blank areas represent those plants for which effects of the particular constituent were not found reported. Table 15 is a summary of the average concentrations of constituents for each stabilization pond recorded. Table 16 is a table of recommended tolerances for trace elements in irrigation water.

Summary of water quality factors

From Table 14, note that concentrations of arsenic as low as 4 ppm have caused damage to certain plants. A value this low coupled with arsenic's tendency to accumulate suggests close scrutiny. The largest average value from Table 15 is 0.004 ppm from Logan. Comparing this value to that specified in Table 16 shows arsenic to be well within the bounds of safety.

Table 14 shows that citrus is extremely sensitive to boron and that soybeans and fruits are fairly sensitive. Sugar beets, cotton, and turfgrass are fairly tolerant. If the effluent is being considered for citrus, the value from Table 16 would bar only that from Manila. For other crops, the Manila boron level would be acceptable.

This study did not define toxic levels of cadmium to crops. However, comparison of Table 15 and Table 16 show that each of the effluents is within the specified limit.

Except for citrus and perhaps fruits, the plants mentioned in this study seem rather tolerant of chloride. If citrus was the crop, the Manila effluent would be the only effluent unsuitable according to the Israeli recommendations (see chloride section). For the other crops mentioned, chloride could be neglected for the effluents under consideration.

A number of plants appear to be rather sensitive to copper with citrus being extremely sensitive. Comparing Table 15 and Table 16 shows the copper values of the effluents to be well within the specified limits.

This study showed that at the pH levels attendant to the pond effluents of Table 15, extremely little if any fluoride would be taken up by the plant and so may be disregarded in this case.

The levels of lead required for toxicity problems are rather high. The concentrations from Table 15 appear to be negligible for the plants listed. Further comparison with Table 16 shows the concentrations in these effluents to be well within the limit.

The only magnesium toxicity problem found was with beans, and the levels required were in excess of 3000 ppm. Table 15 shows the levels of magnesium of each effluent to be at rather low levels in comparison.

The study showed that at the pH levels associated with these effluents, only extremely limited amounts of manganese would be available for plant uptake and therefore manganese would be rather unlikely to be toxic. Also comparison of Table 15 and Table 16 shows each effluent to be well within the specified limits.

The only nitrogen toxicity problem occurring directly was with citrus trees. Simple calculations showed that approximately 210,000 gallons of water would be necessary to supply the 2.5 pounds of nitrogen noted. The calculations used the Huntington level of 6.36 ppm for worst case conditions. This concentration is clearly too small to cause detrimental effects. The indirect effect of summer fertilization on turfgrass is not a problem. The amount of fertilizer recommended for golf courses is about 120 pounds/acre/year and so 3 pounds/acre/year will have negligible effect.

The only case of phosphorus toxicity found was for citrus. The concentration required for injury is 20 ppm which is rather high considering how rapidly phosphorus is tied up in the soil and particularly at the pH levels of the effluents. These effluents should therefore cause no toxicity problems from phosphorus.

The data given for salinity seems rather contradictory. The most restrictive limits found were suggested by the U.S. Salinity Laboratory. (See McKee and Wolfe, 1963.) By these limits Huntington and Manila effluents would be unsuitable.

Levels of only 2.5 ppm of selenium were found to be toxic to certain plants. Comparing Table 15 and Table 16, none of the levels of selenium in the effluents exceeds the specified limit.

The sodium concentration of the Huntington and Manila effluents could not be used for citrus if applied by sprinkler irrigation due to damage caused by foliar absorption of the sodium. Their SAR values also are too high for fruits. Other than these

Table 14. Summary of chemical toxicities for certain plants.[a]

Component	Alfalfa	Barley	Beans	Tomatoes	Potatoes	Radish
Arsenic	4	4				
Bicarbonate & carbonate			600	600	1200	3000
Boron	3	1.5	3	1.75	2	1.5
Cadmium						
Calcium						
Chloride	805	3150	630	1365		
Copper				2	6.4	
Fluoride			100	100		
Iron						
Lead			30		50	
Magnesium			3000			
Manganese			10			10
Nitrogen						
Phosphorus						
Potassium						
Salinity ($EC_e x 10^3$)		16	4	10	10	4
Selenium		2.5				2.5
Silica						
Silver						
Sodium (SAR)	15		15			
Sulfate	480		480	480		
Zinc						

Component	Turf	Beets	Cotton	Soybeans	Peas	Citrus	Oats
Arsenic						5	
Bicarbonate & carbonate							
Boron	10	25	15	5		0.75	1.5
Cadmium		7.25			7.25		
Calcium							
Chloride	665	3500	1750			175	
Copper		6.4				0.1	5
Fluoride						25	
Iron							
Lead		51.8					50
Magnesium							
Manganese		650	81		10		
Nitrogen						2.5No. N/ tree/year	
Phosphorus						20	
Potassium							
Salinity ($EC_e x 10^3$)	18	12	16		6		10
Selenium	2.5						2.5
Silica							
Silver							
Sodium		ESP=15				69	
Sulfate		480	480			480	
Zinc						3	10

[a]Values in ppm unless otherwise specified.

Table 14. Continued.

Component	Corn	Carrots	Wheat	Safflower	Fruits
Arsenic					
Bicarbonate & carbonate					206
Boron	1.5	2	1.5		5
Cadmium					
Calcium					
Chloride	2450		875		455
Copper					
Fluoride					100
Iron					
Lead					
Magnesium					
Manganese	10	1000			
Nitrogen					
Phosphorus					
Potassium					
Salinity ($EC_e x10^3$)	7	7	10		
Selenium			2.5		
Silica					
Silver					
Sodium				ESP=30	SAR=4
Sulfate					
Zinc					

Table 15. Average values for water quality parameters from five stabilization ponds.[a]

Component	Blanding	Huntington	Logan	Manila	Roosevelt
Arsenic	0	0.003	0.004	0.01	0
Bicarbonate	236	538	342	186	379
Boron	0.44	0.48	0.27	1.29	0.72
Cadmium	0	0.003	0.002	0.02	0
Calcium	36.5	209	55	210	45
Carbonate	0.645	3.23	8.0	4.53	2.6
Chloride	47.5	150	47	241	80
Copper	0	0.024	0.01	0.01	0
Fluoride	0.305	0.65	0.27	1.42	0.49
Iron	0.07	0.24	0.15	0.25	0.40
Lead	0.005	0.007	0	0	0
Magnesium	14	268	37	241	49
Manganese	0	0.089	0	0.03	0
Nitrogen	0.55	6.36	2.05	2.90	0.30
pH	7.55	7.73	8.46	8.50	8.10
Phosphorus	10.7	10.9	5.60	0.80	0.20
Potassium	7.5	11.4	9.1	28	14
Salinity (micromhos)	600	4913	648	5118	970
Selenium	0.004	0.011	0.005	0.015	0.01
Silica	16.5	11.57	14.1	3.6	24.0
Silver	0	0.01	0	0.01	0
Sodium	58.5	736	35	826	105
Sulfate	35	2415	17	2631	141
Zinc	0.005	0.04	0.021	0.01	0
RSC (meq/l)	0.88	-23.95	-0.10	-27.3	-0.09
SAR	2.07	7.9	0.89	9.16	2.57

[a] Other than pH and SAR values, units are mg/l unless otherwise noted.

Table 16. Trace element tolerances for irrigation waters.

Element	For Waters Used Continuously On All Soil	For Use Up To 20 Years on Fine Textured Soils of pH 6.0 to 8.5
	mg/l	mg/l
Aluminum	5.0	20.0
Arsenic	0.10	2.0
Beryllium	0.10	0.50
Boron	0.75	2.0-10.0
Cadmium	0.010	0.050
Chromium	0.10	1.0
Cobalt	0.050	5.0
Copper	0.20	5.0
Fluoride	1.0	15.0
Iron	5.0	20.0
Lead	5.0	10.0
Lithium	2.5[b]	2.5[b]
Manganese	0.20	10.0
Molybdenum	0.010	0.050[c]
Nickel	0.20	2.0
Selenium	0.020	0.020
Vanadium	0.10	1.0
Zinc	2.0	10.0

[a] These levels will normally not adversely affect plants or soils. No data available for Mercury, Silver, Tin, Titanium, Tungsten.

[b] Recommended maximum concentration for irrigating citrus is 0.075 mg/l.

[c] For only acid fine textured soils or acid soils with relatively high iron oxide contents.

Source: Above data based primarily on "Water Quality Criteria 1972," National Academy of Science–National Academy of Engineering, Environmental Study Board, ad hoc Committee on Water Quality Criteria, U.S. Government Printing Office (in press).

exceptions, the SAR values are suitable for most crops.

Table 11 grades generally unsuitable, concentrations of sulfate in excess of 576 ppm. Huntington and Manila effluents are totally unsuitable by this standard. The other three effluents rate excellent by the criteria of Table 11.

Zinc levels are sensitive for citrus fruits and so should be checked. Comparison of Table 15 and Table 16 show that each effluent is easily within the specified limit.

Review of Table 15 shows that the criteria of Wilcox et al. (1954) for residual sodium carbonate is easily met. Therefore the bicarbonate/carbonate concentration does not constitute a problem.

Iron, potassium, silica, and silver were found to produce no direct toxic effects. Under certain conditions, however, silica can act to increase the SAR value.

Table 17 shows the effluents and their suggested uses for certain crops. The X's indicate permissible usage. These decisions were based on undiluted effluents. If sufficient dilution were used, those cases which are rejected could become usable. Two ponds, Huntington and Manila, are not recommended for any crop. Both effluents have conductivity values and sulfate concentrations that greatly exceed the recommended tolerance limits. In addition, their SAR values are unsuitable for fruits. Roosevelt's effluent is somewhat questionable for citrus. This is because the boron concentration is at a borderline level. Further research is necessary to refine the results of Table 17 but it does present a starting point.

Land requirement for effluent discharge

Another important consideration is the amount of land required to utilize the total effluent. Since the irrigation requirements for a given crop vary greatly because of differences in soil type, management practices, topography, and climate, a more general approach will be used to determine this question. Table 18 (Overman, 1971) is a computation showing the required land in acres for a given total plant effluent at a number of application rates. For example, if the effluent discharged was 1 mgd and the irrigation rate required by a certain plant was 1 inch/week, then 250 acres would be required to utilize all the effluent. If a new crop were used requiring 2 inches/week, only 125 acres would be needed. This is not a very large area of land and for small communities would be rather typical.

Two implications follow from this calculation. If disposal was the only consideration, a small pasture or other such piece of ground would be easier to acquire than large acreage. If on the other hand, a large number of farmers wanted to use the discharge, only one or two would be able to use the effluent. The solution in this case is obvious–dilution with their normal source of irrigation water. The cost savings would not be as great but there would at least be some reduction. Another advantage might occur. If the effluent had high concentrations of certain components, the dilution factor would assist in making the water less hazardous. This would reduce any fertilization benefits but the overall benefits of

Table 17. Suggested crop usage for five stabilization pond effluents.

Crop	Blanding	Huntington[a]	Logan	Manila[a]	Roosevelt
Alfalfa	X		X		X
Barley	X		X		X
Beans	X		X		X
Tomatoes	X		X		X
Potatoes	X		X		X
Radish	X		X		X
Turfgrass	X		X		X
Beets	X		X		X
Cotton	X		X		X
Soybeans	X		X		X
Peas	X		X		X
Citrus	X		X		b
Oats	X		X		X
Corn	X		X		X
Carrots	X		X		X
Wheat	X		X		X
Safflower	X		X		X
Fruits	X	c	X	c	X

[a] Rejected for all crops because of high salinity and sulfate values.

[b] Questionable because of borderline boron value.

[c] Rejected for fruits because of high SAR value.

waste disposal, less expensive irrigation water, and reduced toxicity potential would be well worth the loss.

Case histories

Sewage effluent has been used successfully in many trials. Day and Tucker (1960) reported successful results with small grains. They grew Arivat barley, Palentine oats, and Ramona 50 wheat using flood irrigation. The water used for irrigation was activated sludge effluent and contained approximately 65 pounds nitrogen, 50 pounds P_2O_5, and 32 pounds K_2O per acre foot. Four treatments were used: (1) Pump water with no fertilizer added; (2) pump water with the recommended fertilizer rates (100 pounds N, 75 pounds P_2O_5, and 0 pounds K_2O/acre); (3) pump water with synthetic sewage (200 pounds N, 150 pounds P_2O_5, and 100 pounds K_2O/acre); and (4) sewage effluent with no additional fertilizer. Results of the experiment are shown in Table 19. Note that the yield for oats and wheat was greater using sewage effluent than for any other treatment. Barley did not perform as well using sewage effluent. The authors speculated that the greater salinity and surfactant content may have been the reason.

Another project using sewage effluent was a golf course in Allamuch, New Jersey (Keshen, 1971). This course was able to obtain wastewater at no cost

Table 18. Land requirement for sewage effluent disposal as irrigation water in acres (Overman, 1971).

Application Rate (inches/week)	Discharge Rate, mgd		
	0.1	1	10
½	50	500	5000
1	25	250	2500
2	13	125	1250
3½	7	72	720
7	4	36	360
14	2	18	180
28	1	9	90

Table 19. Results of four different treatments on small grains.

Treatment	Tons/acre		
	Barley	Oats	Wheat
1. Pump water, no fert. added	2.42	2.13	2.76
2. Pump water, recom. fert. added	5.64	2.73	5.47
3. Pump water, synthet. sewage	7.15	4.27	5.93
4. Sewage effl., no added fert.	5.88	6.05	6.43

which resulted in a savings of nearly $21,000 a year. They also felt that because of the nutrients in the water, that money was being saved in decreased commercial fertilizer needs. Thus it seems that sewage effluent would be ideal for golf courses. If the nitrogen concentrations were as high as 30 ppm or greater as nitrate, there are some potential hazards that should be considered however. Studies (Zanoni et al., 1967) have indicated that turf should not be fertilized during the summer. The primary reason is that above a soil temperature of 50°F or 60°F, nutrients are utilized by plants to promote top growth. This causes the grass to become soft and succulant and therefore most attractive to disease and insects. This problem is further emphasized because golf courses in general irrigate at night resulting in warm, moist areas ideal for disease habitats. In contrast, when fertilization is performed only in the early spring and not during the summer, the grass tops are not soft and succulant and so are much less susceptible to insects and disease.

In addition, besides running the risk of losing turf to insect and disease damage, the cost for maintenance will increase because mowing would be required more frequently and larger amounts of pesticides would be needed. The increased usage of pesticides would of course tend to offset the savings in commercial fertilizer cost reductions. Then too, this increased usage contributes more pesticides to the ecology which has already become a major area of concern.

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APPENDIX

Table 20. Analysis of Blanding and Roosevelt pond effluents.[a]

Component	Blanding		Roosevelt
	11/25/69	8/19/70	12/15/70
Conductivity (μmhos/cm)	680	520	970
pH	7.85	7.25	8.10
TDS	468	364	710
Total alkalinity	228	160	314
Arsenic	0	0	0
Barium	0	0	0
Bicarbonate	277	195	379
Boron	0.38	0.50	0.72
Cadmium	0	0	0
Calcium	40	33	45
Carbonate	1.1	0.19	2.6
Chloride	40	55	80
Copper	0	0	0
Fluoride	0.55	0.06	0.49
Total hardness	148	144	312
Total iron	0	0.14	0.40
Lead	0.01	0	0
Magnesium	13	15	49
Manganese	0	0	0
Nitrate	1.0	0.1	0.3
Phosphorus	12.6	8.8	0.2
Potassium	5	10	14
Selenium	0.008	0	0.01
Silica	16	17	24
Silver	0	0	0
Sodium	37	80	105
Sulfate	30	40	141
Zinc	0	0.01	0

[a] All units mg/l unless otherwise specified.

Table 21. Analysis of Huntington pond effluent.[a]

Component	Date of Reading						
	7/10/68	11/13/68	3/19/69	7/21/69	11/24/69	4/14/70	8/18/70
Conductivity (μmhos/cm)	4790	9800	10200	850	2750	3000	3000
pH	7.55	8.3	8.1	7.2	7.35	7.2	8.4
TDS	4510	9532	8865	478	2380	2532	2632
Total alkalinity	417	690	560	288	382	395	396
Arsenic	0.01	0	0.01	0	0	0	0
Barium	0	0	0	0	0	0	0
Bicarbonate	507	824	674	351	464	480	469
Boron	0.44	0.70	0.52	0.60	0.33	0.28	0.47
Cadmium	0	0	0	0	0	0.01	0.01
Calcium	240	360	320	45	144	151	200
Carbonate	0.99	9.1	4.7	0.31	0.57	0.42	6.50
Chloride	140	270	342	50	80	80	85
Copper	0.02	0.08	0.05	0	0.02	0	0
Fluoride	0.58	1.06	1.03	0.23	0.78	0.30	0.57
Total hardness	1604	3200	3228	244	976	954	1175
Total iron	0.90	0.10	0.14	0.20	0	0.20	0.16
Lead	0	0	0.05	0	0	0	0
Magnesium	243	559	590	32	150	140	164
Manganese	0.10	0.18	0.09	0	0.01	0.10	0.14
Nitrate	0.20	1.00	0.70	35.0	3.3	3.5	0.80
Phosphorus	5.6	14	3.1	27.6	5.4	12.4	7.9
Potassium	15	14	14	11	7	8	11
Selenium	0.01	0.02	0.02	0	0.005	0.01	0.01
Silica	11	20	10	11	9	8	12
Silver	0	0	0	0	0	0.01	0
Sodium	760	1700	1600	60	330	375	330
Sulfate	2512	5520	5680	64	422	1319	1387
Zinc	0	0.04	0.20	0.02	0	0.02	0

[a] All units mg/l unless otherwise specified.

Table 22. Analysis of Logan pond effluent.[a]

Component	Date of Reading						
	9/11/68	1/24/69	5/21/69	9/24/69	3/11/70	6/3/70	11/14/70
Conductivity (μmhos/cm)	635	710	710	660	510	670	565
pH	8.15	7.9	8.25	8.8	8.9	7.85	8.95
TDS	420	430	402	424	356	442	396
Total alkalinity	301	318	271	324	262	310	297
Arsenic	0	0	0.02	0	0	0	0
Barium	0	0	0	0	0	0	0
Bicarbonate	361	384	324	369	294	375	336
Boron	0.29	0.26	0.24	0.27	0.21	0.31	0.26
Cadmium	0	0	0	0.005	0	0.01	0
Calcium	57	59	57	60	43	62	58
Carbonate	2.8	1.7	3.2	12.8	12.9	1.5	13.1
Chloride	36	44	58	43	42	50	40
Copper	0	0.03	0.01	0	0	0.01	0.02
Fluoride	0.19	0.30	0.29	0.28	0.28	0.29	0.23
Total hardness	300	310	271	320	260	307	292
Total iron	0.90	0.10	0	0	0	0.20	0
Lead	0	0	0	0	0	0	0
Magnesium	38	40	31	41	37	37	36
Manganese	0	0	0	0	0	0	0
Nitrate	6.9	3.7	2.8	0.7	0.2	1.7	0.2
Phosphorus	6.2	1.2	4.7	6.2	5.5	10.7	5.3
Potassium	10	10	9.0	10	7.0	10	8.0
Selenium	0	0.01	0	0	0.01	0	0.02
Silica	13	18	13	20	2	16	17
Silver	0	0	0	0	0	0	0
Sodium	30	35	37	31	30	36	24
Sulfate	17	21	19	15	17	14	14
Zinc	0	0.12	0.03	0.01	0	0	0

[a] All units mg/l unless otherwise specified.

Table 23. Analysis of Manila pond effluent.[a]

Component	Date of Reading					
	5/27/68	6/12/69	10/28/69	3/31/70	8/4/70	12/15/70
Conductivity (μmhos/cm)	5350	5140	6760	2910	5050	5500
pH	8.0	9.25	7.4	8.85	8.7	8.6
TDS	5575	4688	6724	2440	5100	5600
Total alkalinity	178	101	100	196	222	158
Arsenic	0	0	0.01	0	0	0
Barium	0	0	0	0	0	0
Bicarbonate	215	102	122	231	259	185
Boron	1.2	1.0	1.7	0.85	1.5	1.5
Cadmium	0	0	0	0	0.02	0
Calcium	265	158	232	152	232	221
Carbonate	1.2	10.1	0.17	4.5	7.1	4.1
Chloride	260	233	346	111	225	270
Copper	0.02	0	0	0	0.01	0.03
Fluoride	1.6	1.16	1.68	1.18	1.4	1.48
Total hardness	16[illegible]	1285	1892	905	162[illegible]	1710
Total iron	0.3[illegible]	0	0.80	0.06	0.26	0.10
Lead	0	0	0	0	0	0
Magnesium	247	217	319	128	253	282
Manganese	0	0	0	0.20	0	0
Nitrate	7.6	0.4	1.7	4.0	3.0	0.7
Phosphorus	0.10	0.2	0	3.6	0.8	0.1
Potassium	30	21	29	20	32	35
Selenium	0.01	0.03	0	0.01	0.02	0.02
Silica	1.0	0.8	1.0	11	5.0	3.0
Silver	0	0	0	0.01	0	0
Sodium	1000	800	1150	340	750	915
Sulfate	3258	2700	2450	1350	2830	3200
Zinc	0.01	0	0	0	0	0

[a] All units mg/l unless otherwise specified.

Table 24. Guidelines to levels of toxic substances in drinking water for livestock.[a]

Constituent	Upper Limit	Constituent	Upper Limit
Aluminum (Al)	5 mg/l	Lead (pb)	0.1 mg/l[b]
Arsenic (As)	0.2 mg/l	Manganese (Mn)	no data
Beryllium (Be)	no data	Mercury (Hg)	0.01 mg/l
Boron (B)	5.0 mg/l	Molybdenum (Mo)	0.5 mg/l
Cadmium (Cd)	0.05 mg/l	Nitrate & Nitrite (NO_3-N+NO_2-N)	100 mg/l
Chromium (Cr)	1.0 mg/l	Nitrite (NO_2-N)	10 mg/l
Cobalt (Co)	1.0 mg/l	Selenium (Se)	0.05 mg/l
Copper (Cu)	0.5 mg/l	Vanadium (Va)	0.10 mg/l
Fluoride (F)	2.0 mg/l	Zinc (Zn)	25 mg/l
Iron (Fe)	no data	Total Dissolved Solids (TDS)	10,000 mg/l[c]

[a] Based primarily on "Water Quality Criteria 1972," National Academy of Science–National Academy of Engineering, Environmental Study Board, ad hoc Committee on Water Quality Criteria, U.S. Government Printing Office (in press).

[b] Lead is accumulative and problems may begin at threshold value = 0.05 mg/l.

[c] Guide to the use of saline waters for livestock and poultry.

UPGRADING LAGOON TREATMENT WITH LAND APPLICATION

R. E. Thomas[1]

Introduction

The enactment of PL 92-500 and subsequent actions have necessitated a critical evaluation of processes which can be utilized to upgrade lagoon treatment of wastewaters. Land application is an approach which shows promise because it embodies the principle of water reuse for conservation and addresses the "no discharge of pollutants" goal of PL 92-500. The concept of applying lagoon effluents to the land is not new and there are many examples of such systems in the United States. Most of these existing systems are located in water-short regions and the reason for utilizing this wastewater management technique has been the conservation of a short water supply and/or the disposal of a pond effluent where a surface water discharge was not convenient or appropriate. A long history of continued use at many locations indicates that the land application concept has been reasonably successful in achieving the intended goals of water conservation or convenient disposal in the absence of access to a suitable surface water discharge. The existence of such systems also provides a ready laboratory for assessing the performance of several land application alternatives for upgrading lagoon treatment in light of the needs of today and the future.

Upgrading of lagoons can also be achieved by land application techniques which are not in common use at the present time since the options available are many. These options include addition of one of several land application approaches or complete replacement of the lagoon system by a land application approach. This wide range in available options stems from the fact that there are three basic land application approaches which have potential for use in the upgrading of lagoon treatment. These land application approaches are frequently designated as (1) crop irrigation, (2) infiltration-percolation, and (3) overland flow. Each of these approaches has unique characteristics for adaptation to site conditions and treatment needs.

The purpose of this presentation will be to describe the use of several of the many ways in which land application approaches are or can be utilized to upgrade lagoon treatment. The discussion will include developing technology as well as examples of land application approaches which have been in use for many decades.

Some Practical Options

The terminology of "soil treatment" or perhaps more appropriately "land application for treatment and/or reuse" is liberally sprinkled with buzz words such as "the living filter." This situation behooves one who is discussing the topic to acquaint his audience with his own terminology. I am a proponent of the terminology base which divides land application approaches into three classes, as follows: (1) Crop irrigation, which is characterized by comparatively low application rates (less than 10 cm per week) and emphasizes the reuse of wastewater for beneficial growth of vegetation; (2) overland flow, which is characterized by intermediate application rates (7.5 to 15 cm per week) and emphasizes treatment with an effluent discharge to surface waters; and (3) infiltration-percolation, which is characterized by high application rates (up to 150 cm per week) and emphasizes treatment with underground storage of the reclaimed wastewater. With the terminology base established, some practical options can be addressed for implementing current land application approaches to upgrade lagoon treatment. Crop irrigation as an add-on to existing systems and infiltration-percolation as an add-on to existing systems in this category will be discussed.

Crop irrigation following lagoon treatment

Crop irrigation following lagoon treatment or polishing lagoons has been practiced as a commonly accepted waste management technique for many decades. There are several hundred of these facilities currently operating in the southwest and far west. It is difficult to find a single source which identifies most of these systems, but there are several sources which provide information on the location, age, design, and operating schedule for many of these systems. Deaner (1971) reports on 132 reclamation operations in California including identification of the treatment processes preceding land treatment. About 70 percent of the systems have polishing lagoons following a variety of pretreatment processes,

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some 23 percent follow activated sludge or trickling filters with direct land application, and 7 percent have land application following primary treatment. Crop irrigation is practiced at about 75 percent of the sites, while landscape irrigation or golf course irrigation is practiced at 25 percent of the sites. Another source of information on crop irrigation following lagoon treatment is the EPA Municipal Waste Inventory. This source provides information on some 160 facilities with lagoon treatment followed by crop irrigation. Sullivan, Cohn, and Baxter (1973) report detailed information on 26 sites visited during their survey of land application facilities. Eighteen of these systems included oxidation ponds and crop irrigation as the last two steps in the overall treatment process. Three of the systems visited during this survey had been in operation for over 50 years and an additional 11 systems had been in operation for 10 to 50 years. Myers and Williams (1970) report on the design of lagoon systems followed by crop irrigation in Michigan. They indicate that several were in operation in 1970 and all of the 15 systems which they were planning for construction in 1971 would involve irrigation with the treated effluent. Dean (1974) has surveyed the use of land application in the Rocky Mountain-Prairie region and reports 10 facilities where lagoon treatment is followed by land application. One of these systems is 10 years old, while all of the others have been placed in service since 1972. This cursory look at current practices shows that crop irrigation following lagoon treatment is a reality under widely differing climatic conditions. It is obvious that these existing systems are sources of design information for use in other locales as well as a ready laboratory for assessing the performance of current designs in relation to the objectives of PL 92-500.

Design of crop irrigation systems

Myers and Williams (1970) give general application rates ranging from 0.3 to 0.8 cm per hour programmed to achieve weekly rates of 2.5 to 7.5 cm and yearly rates of 125 to 250 cm. They point out that irrigation system designs are site specific and make the following analogy, "Just as one does not hire a butcher when he needs a surgeon, one should not hire a plumber when he needs a professional irrigation engineer." Pound and Crites (1973) analyze the data collected by Sullivan, Cohn, and Baxter (1973) as well as information from site visits which they conducted during their study on current design and operating information. In their chapter on irrigation with municipal effluents, they present details on seven facilities with lagoon treatment and crop irrigation as the final treatment steps. They defined crop irrigation to include application rates of 10 cm per week or less, and divided the facilities discussed into high irrigation (7.9 to 10.7 cm per week), moderate irrigation (5.6 to 7.6 cm per week), and low irrigation (0.8 to 3.8 cm per week).

Information contained in reports such as those by Pound and Crites (1973), Myers and Williams (1970), and Sullivan, Cohn, and Baxter (1973), demonstrate that crop irrigation systems have been designed to adequately fulfill a wide range of local climatic conditions and institutional constraints. In many instances, these existing systems have evolved through long periods of community growth and associated problems. Long-term experience at these existing systems has not, as yet, generated a ready design manual, but persons contemplating the upgrading of lagoon treatment by addition of crop irrigation can glean valuable design, operating, and cost information from this reservoir of information on existing systems.

Infiltration-percolation following lagoon treatment

Infiltration-percolation following lagoon treatment is another practice which has been widely accepted for many decades. There are many such systems in the United States but it is even more difficult to identify and locate these systems than it is to identify and locate crop irrigation systems. Poor definition of terminology in the past has led to the inclusion of infiltration-percolation systems under the broad category of crop irrigation approaches. Regardless of this limitation, there are several sources which indicate the extent to which the infiltration-percolation approach has been utilized as a final step in wastewater management. Deaner (1971) identifies seven California systems as groundwater recharge (infiltration-percolation) operations, two of which include lagoon treatment prior to the recharge operation. Information from the EPA Municipal Waste Inventory differs from Deaner's (1971) interpretation of the use of infiltration-percolation in California. A 1972 listing from the inventory with a total of 64 California facilities (less than 50 percent of Deaner's total) lists 21 facilities as infiltration-percolation type systems. Discrepancies of this nature are readily attributable to the lack of uniform definition for terminology. Overall, the listing from the Municipal Waste Inventory indicated that about 15 percent of 160 lagoon treatment facilities utilizing land application were entered as infiltration-percolation systems. Sullivan, Cohn, and Baxter (1973) noted that they did not visit six facilities initially selected for survey because the systems were percolation systems. Pound and Crites (1973) noted two cropping systems with application rates greatly exceeding their defined upper limit of 10 cm per week for crop irrigation and classified these systems as infiltration-percolation systems. Only one of the infiltration-percolation systems which they discuss in depth involves a lagoon in the treatment process.

Design of infiltration-percolation systems

It is obvious from the foregoing discussion that infiltration-percolation treatment of lagoon effluents is less common than crop irrigation with pond effluents. Consequently, existing systems do not provide a broad based reservoir of design, operating, and cost information. Persons contemplating the upgrading of lagoon treatment by addition of infiltration-percolation treatment will find that information from closely related studies on infiltration-percolation of activated sludge effluents and trickling filter effluents may be the best information available to them. Thomas and Harlin (1972) describe some research studies which provide quantitative data under controlled operating conditions. Aulenbach et al. (1973) report the results of intensive studies on a 35 year old facility in a cool, humid climate with an application rate of about 50 cm per week. Pound and Crites (1973) evaluate the design and management of several operational systems with application rates ranging from 20 to 200 cm per week.

Many reserachers are continuing the assessment of performance by existing infiltration-percolation systems; therefore, the research community is an important source of recent information on system designs and system performance.

A Developing Land Application Approach

There is a newly developing land application approach which shows promise as a wastewater management process for utilization on slowly permeable soils. This approach is the overland-flow system which has been used with excellent results by the food processing industry. Current research is directed to the development of this technique for advanced waste treatment of secondary effluents and, alternately, to the development of this technique for complete treatment of raw domestic wastewater.

Overland-flow as an add-on to secondary treatment

Hoeppel, Hunt, and Delaney (1973) are conducting studies on the use of the overland-flow approach for advanced treatment of secondary effluents. The results of their short-term greenhouse studies show that characteristic total nitrogen removals of more than 80 percent were achieved using secondary effluent amended with sucrose to achieve a chemical oxygen demand of 200 mg/l at an application rate of 6.35 cm per week. A second experiment with sludge amended soil versus a control soil showed that sludge addition neither improved nor hindered the removal of total nitrogen. These preliminary studies also showed that several heavy metals were effectively removed from the wastewater as it moved downslope.

The encouraging results of these bench-scale studies indicate that the overland-flow technique shows promise as an advanced waste treatment process with low energy requirements and no additional sludge production. As such, it may be a candidate for upgrading lagoon treatment where additional land is available for use in the treatment process.

Overland-flow as an alternate to lagoon treatment

Thomas et al. (1974) are conducting studies on the treatment of raw domestic wastewater by the overland-flow approach. The results of 18 months of pilot scale field studies show that the overland-flow process produces an effluent substantially better in quality than the effluent from an activated sludge plant. The capability of overland-flow to maintain this level of performance during year-round operation at an average application rate of about 10 cm per week indicates that total land requirements for overland-flow treatment would be comparable to land requirements for multiple cell lagoon systems.

The pilot-scale field study is now in its third year and the overland-flow process continues to produce an effluent with the following composition: Suspended solids and biochemical oxygen demand of less than 10 mg/l; total nitrogen ranging from 2.5 mg/l in summer to 7 mg/l in winter; and total phosphorus of 4 to 5 mg/l. The total phosphorus can be reduced to less than 1 mg/l by addition of a precipitant such as aluminum sulfate.

It is obvious that overland-flow is capable of achieving advanced treatment of raw domestic sewage on a year-round basis with favorable climatic conditions. We are now initiating a full-scale (0.1 mgd) development study to evaluate performance under operating conditions expected for small sewage treatment facilities. The results of this first full-scale study will be complemented by similar studies under a variety of climatic conditions in order to ascertain obvious constraints due to severe winter weather and other site specific factors.

Design of overland-flow systems

The basic design of overland-flow systems which has been developed for treatment of food processing wastewaters will apply to systems treating domestic wastewaters. A soil with restricted permeability is a prerequisite common to all overland-flow systems, as is the leveling of the land to obtain uniform and gentle slopes. Construction of interceptor terraces at about 60 m spacings and the construction of a distribution system (usually automated) are also integral features of all overland-flow facilities. Daily application and drying cycles with regular 1- or [illegible]-day drying cycles at 5- or 6-day intervals will also be required to maintain a microenvironment which maintains treatment performance without promoting the breeding of nuisance insects.

Major questions to be elaborated regarding technical aspects of system design are the rate of application and constraints placed on the length of the operational season by climatic conditions. The need for elaborating application rates is readily demonstrated by comparing the application rates used in the two research studies which have been discussed previously. Thomas et al. (1974) observed successful treatment of raw domestic sewage at an application rate 1.5 times as great as the application rate that Hoeppel, Hunt, and Delaney (1973) used for applying amended secondary effluent. The need to ascertain the constraints imposed by subfreezing winter temperatures are complex and many. Resolution of these constraints will require evaluation of the overland-flow approach at many field sites with varying climatic conditions.

Summary

Land application is a viable approach for upgrading lagoon treatment to meet newly imposed standards for secondary treatment. It is also a viable approach for satisfying even more stringent requirements which are proposed for future implementation. There are several land application alternatives which can be considered, and the alternative selected will depend largely upon local requirements and site characteristics.

Crop irrigation is an alternative which has been in use for many decades under a variety of climatic conditions with a high degree of success. Existing crop irrigation systems provide a vast reservoir of information on design and operating experiences which have led to problems, as well as successes. This reservoir of information from existing systems is a valuable resource which can be utilized by those contemplating crop irrigation for upgrading lagoon treatment.

Infiltration-percolation is another alternative which has limited but widespread use for advanced waste treatment of secondary effluents. It is more typical for infiltration-percolation to follow activated sludge or trickling filters as the secondary treatment process, therefore, the reservoir of existing information on design and operating experience following lagoon treatment is not extensive. Information on infiltration-percolation following other conventional processes for achieving secondary treatment does provide a valuable resource which can be effectively utilized by those contemplating infiltration-percolation for upgrading lagoon treatment.

Current research studies are exploring the utility and practicality of utilizing other land application alternatives for management of domestic wastewaters. Overland-flow is a land application alternative which is being evaluated for upgrading secondary effluents and for complete replacement of conventional secondary treatment processes. Development of both of these approaches is at a rudimentary stage but they show promise for successful implementation in practical situations.

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LAGOON EFFLUENT SOLIDS CONTROL BY BIOLOGICAL HARVESTING

W. R. Duffer[1]

Introduction

Stabilization of organic wastes in lagoons has developed as a wastewater treatment which favors establishment and maintenance of specific types of organisms. Planktonic algae in combination with aerobic bacteria are the organisms responsible for oxidation of organic materials. Since a large proportion of the organic materials are converted to phytoplankton, suspended solids are usually present in relatively high concentrations in lagoon effluents. Research effort for this wastewater treatment process has centered around design and operation practices for improving production of algae, rather than developing design criteria for enhancing establishment of organismal populations to consume the excessive algal production.

It is the purpose of this paper to (1) review the accomplishments of several studies oriented toward removal of phytoplankton by other aquatic organisms, (2) identify some of the problems associated with biological harvesting of phytoplankton, and (3) suggest research areas which should be developed.

Suspended Solids Removal

Studies which relate to solids removal have been conducted in both marine and freshwater. It should be remembered at this point, however, that the concept of control of algae by organisms in the marine and freshwater food chains is not completely proven and has not gained acceptance as a treatment practice due to the uncertainties involved. Although most of the experiments conducted in the United States have been small scale, several have been highly successful and will provide a sound basis for development of applied pilot scale research efforts. Investigations have centered around several types of organisms including zooplankton, bivalve mollusks, and fish.

Zooplankton

Several field and laboratory experiments have been conducted which utilize the cladoceran, *Daphnia*, for removing excessive algal growth. A two-stage pond system was established at Calabasas, California, for polishing activated sludge effluent (Las Virgenes Municipal Water District, 1973). The shallow first-stage pond developed a high concentration of phytoplankton. In the second-stage pond, which had a water depth of about 3 meters, a population of *Daphnia pulex* effectively removed the algae. The capacity of the first stage was about three times greater than that of the second stage and the system was operated with about 10 days' detention in each stage. *Daphnia* concentrations above 500 organisms/liter were responsible for a suspended solids reduction of about 50 percent. A summer decline of the *Daphnia* population and occasional invasion of *Daphnia* or rotifers in the first-stage pond were major problems encountered.

Aquarium experiments employing *Daphnia* and *Lemna* cultures were conducted in Israel (Ehrlich, 1966). In one experiment, sewage stabilization pond effluent was stored in a feeding container and allowed to drip into nine basins at varying rates. Each basin contained *Lemna-Daphnia* cultures and the periods of detention ranged from 8 to 360 hours. Effluent from the experimental basins was relatively clear, while samples from the feeding container remained turbid. In another experiment, stabilization pond effluent was fed to four aquaria, each having a detention period of 10 days. This test was designed to determine the relative roles of *Lemna* and *Daphnia* in the suppression of a population of *Chlorella*. One unit contained a *Lemna-Daphnia* culture, another *Lemna* only, another *Daphnia* only, and the fourth was maintained as a blank. Test results indicated that the *Lemna-Daphnia* unit gave the best reduction of *Chlorella*, followed by the *Lemna* only unit. Numbers of *Chlorella* per cm^3 were similar for the *Daphnia* only unit and the blank following 13 days of operation.

Results of a survey of domestic wastewater treatment facilities in Texas utilizing ponds indicate that occurrence of *Daphnia* is quite uncommon (Dinges, 1974a). Only 29 of the 470 pond systems support *Daphnia* populations. These systems, however, are noted to maintain relatively stable conditions and to have clear effluents. In a discussion of the environmental requirements of *Daphnia*, Dinges and Rust (1972) conclude that photo-period is the dominant environmental factor affecting *Daphnia* in stabilization ponds. In general, population pulses in Texas ponds appear when the period of sunlight approaches about 10 hours per day and remain until the period of sunlight approaches 14 hours per day. Other environmental factors affecting pulses include hydrogen ion concentration, presence of dissolved oxygen, free ammonia and hydrogen sulfide.

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An experimental *Daphnia* culture pond having a capacity of 10,000 gallons was constructed to treat effluent from sewage oxidation ponds at Giddings, Texas (Dinges, 1973). The site was selected because the ponds had a past history of producing abundant algae and had never been known to support *Daphnia* populations. The experimental pond had a surface area of .01 acre and was operated for an 11-day detention period. The investigation was conducted in two phases for three months during the minimum seasonal photo-period. The first phase was designated to determine the effectiveness of pH control by shading to reduce algal growth, and the second phase evaluated pH control through chemical addition and sulfide reduction by aeration. A continuous culture of *Daphnia* was maintained during both phases and suspended solids reductions for the stabilization pond effluent were 86 percent and 83 percent, respectively.

Dinges (1974b) has proposed several sewage stabilization designs for separate facilities which take into account environmental requirements of zooplankters. Design considerations include pond site selection, construction of berms and baffles, inlet and outlet structures, mixing and depth, substrate and pH regulation. Culture ponds requiring rigid control and extensive management are considered, as well as primative installations requiring minimum regulation.

Bivalve mollusks

Both marine and freshwater species of mollusks have been cultured to remove algae from suspension. *Corbicula*, an oriental freshwater clam, has been reported to filter in excess of 0.5 liters per day per organism (Prokopovich, 1969). Greer and Ziebell (1972), in studies designed for removal of orthophosphate from water, utilized natural algal populations and clam filtration. *Corbicula* could clarify water containing high concentrations of algae and survive under highly enriched conditions, providing water was circulated and temperatures maintained below 30°C. *Corbicula* has a very high reproductive capacity and does not require an obligate parasitic stage in its life cycle.

Several laboratory experiments have been performed by a group at Woods Hole Oceanographic Institution using oysters and other marine bivalve mollusks for removal of algae which was grown in a combination of secondarily treated wastewater and seawater (Rytler, 1973). These experiments were highly successful from the standpoint of nutrient removal, and a prototype process was developed around the food web concept. Several growth systems involving marine phytoplankton, oysters, deposit feeders, and seaweed were combined in series and fed secondarily treated wastewater diluted with filtered seawater. Filter-feeding bivalve herbivores removed 85 percent of the algae fed to the system. In order to determine performance of the multi-species food web concept on a large scale, pilot plant facilities have been designed and constructed at Woods Hole (Huguenin, 1974).

Fish

In Asia, fish culture in highly enriched waters has been practiced for centuries and the most commonly cultured fish are members of the carp family (*Cyprinidoe*) (Bardach et al., 1972). In China, the most commonly cultured phytoplankton feeder is the silver carp. The Chinese, however, usually stock several types of fish in culture ponds. This practice insures the most efficient use of the variety of fish food organisms available.

The Arkansas Game and Fish Commission has obtained the silver carp on an experimental basis and most of the effort to date has centered around propogation studies (Husley, 1974). State fish hatchery personnel have successfully spawned the silver carp and indicate that aquarium water containing high concentrations of algae could be cleared by this plankton feeding species within 24 hours. Hatchery biologists expect to expand their pond culture studies to include native fish species in a polyculture system.

Fathead minnows were stocked and successfully cultured in the sewage stabilization system at Belding, Michigan (Trimberger, 1972). The system consisted of five oxidation ponds operated in a series. Minnows were placed in the last three ponds of the series at stocking rates less than one pound per acre. Six months after stocking, the fish were harvested, yielding 378 pounds per acre. Ponds stocked with minnows were noted to be less turbid than the other oxidation ponds.

A field study designed for removal of algae from sewage oxidation ponds through the food chain mechnanism was conducted by the Oklahoma State Department of Health (Coleman et al., 1974). A six-cell lagoon system receiving raw domestic wastewater was operated in series with the first two cells being aerated. Fingerling channel catfish were stocked in Cells 3 and 4, while golden shiner adults were stocked in Cells 5 and 6. Fathead minnows and *Talapia nilotica* were also placed in the third cell. Lagoons were contaminated with the black bullhead, green sunfish, and mosquito fish already present in the system. Fish biomass estimates were based on seine hauls from a closed area of each cell. *Talapia* biomass increased from an initial 4 to 163 pounds during a period of about four months. Golden shiner minnow biomass increased from 85 to 535 pounds during a period of about four months. The biomass of channel catfish increased from an initial 600 to an estimated 4,400 pounds in a period of about eight weeks. Mean values of weekly analyses for suspended solids during the 4-month period of fish culture indicate a general decrease through the pond series with an 83 percent solids reduction by comparing the effluents of Ponds 2 and 6.

Problem Areas and Research Needs

Based on the number of successful investigations utilizing a variety of organisms for removal of high concentrations of algae, the food chain concept appears to be a basically sound approach for development of a mechanism for solids control in sewage oxidation ponds. However, it should be pointed out that the greatest effort to present has consisted of either small-scale laboratory experiments or descriptive studies in large systems where only limited control was possible. Carefully controlled pilot-scale experiments are required prior to full-scale demonstrations in order to assure research flexibility for determining design criteria and management approaches.

Specific areas which require additional research effort for development of biological control of lagoon solids are:

1. Extent of selective solids uptake by various species of herbivores, as well as the mechanisms involved, should be established.

2. A greater number of herbivore species should be examined to determine the compatibility of their environmental requirements with wastewater effluents.

3. The science of ecology must be applied to increase the efficiency and stability of biological control systems. Engineering design parameters should be couched in terms of the multi-species approach and polyculture of organisms rather than the capability of single species.

4. Seasonal aspects of treatment culture units and life stage requirements of organisms must be taken into account.

5. Values must be assigned to productivity of culture systems in terms of potential for use in aquaculture and commercial markets.

6. Potential for problems to areas such as recreation, agriculture, and aquaculture, must be determined for introduction of exotic species such as *Corbicula* or the silver carp.

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PROGRESS REPORT: BLUE SPRINGS LAGOON STUDY

BLUE SPRINGS, MISSOURI

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Introduction

Public Law 92-500, the Federal Water Pollution Control Amendments, was enacted by Congress on October 18, 1972. These amendments provided the necessary regulatory environmental legislation for controlling municipal and industrial waste sources prior to discharge to receiving waters. The control is exercised through the implementation of the National Pollutant Discharge Elimination System (NPDES) as outlined in Section 402 of the amended act. Simply stated, the NPDES requires the issuance of permits for every discharge to navigable waters.

These permits are written to include implementation schedules for achieving specified levels of waste treatment and prescribed self monitoring requirements. In the case of municipal waste discharge permits, the permits require secondary treatment on all discharge by 1977. For permit conditions, secondary treatment is defined by concentration limits on 5-day biochemical oxygen demand (BOD_5) and nonfilterable solids (suspended solids or NFS) and fecal coliform densities. The limits are 30 mg/l, 30 mg/l, and 200 organisms/100 ml, respectively.

The economic base of Region VII of the Environmental Protection Agency is primarily agricultural. As a consequence, approximately 80 percent of the waste treatment facilities are located in small rural communities which are, for the most part, declining in population with an attendent reduction in resources to provide community services. In many instances, these small towns are not constrained by land availability and have resorted to lagoon systems for municipal waste treatment. The towns usually have a locker plant which does some custom slaughtering and/or a creamery which contribute shock loads to the sewage system. The prime question is, will a lagoon system furnish the necessary degree of treatment to meet the secondary criteria?

Over the past 3 years we have sampled approximately 330 lagoon systems of various sizes and configurations. This field sampling exercise has been conducted over all seasons and is usually comprised of one to three 24-hour composite samples of the influent and a similar number of grab samples from the effluent. During the same period, we have also sampled a number of small mechanical and/or package plants. In March of this year, a summary of our plant experience was compiled to provide a general assessment of plant performance. These data are shown in Table 1.

Average waste treatment performance for BOD_5 and suspended solids removal to meet secondary treatment requirements occurred only in three-cell lagoon systems. Average fecal coliform densities in effluents from all systems exceeded the secondary treatment definition limit. Since all of the data presented were collected in a more or less random fashion during various seasons, the resulting averages should reflect normal operating conditions. These data then provide a basis for developing practical plans for upgrading waste treatment for the immediate future.

There are many economic implications involved in selecting waste treatment systems. One of the prime concerns is eligibility for a Federal Construction grant under the requirements of meeting secondary treatment. A special intensive study was planned to evaluate seasonal performance of a three-cell lagoon to provide data on the ability of these systems to meet secondary treatment criteria.

The Blue Springs, Missouri, three-cell in a series lagoon system was selected for the study. This system, comprised of three cells with an area of 16.8, 5.0, and 1.7 hectares (41.5, 12.4, and 4.1 acres) respectively, was designed to serve a population equivalent of [illegible],000 and is now serving an estimated 12,000 to 13,000 people.

Study Description

A study plan was developed with three objectives in mind:

1. Determination of the performance of a three-cell, series operated, lagoon.

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Table 1. Treatment plant effluent data, Region VII.

Type of Plant	No. of Plants	BOD_5 mg/l	Suspended Solids mg/l	Fecal No./100 ml Coliform
		Ave. Range	Ave. Range	Ave. Range
1 Cell Lagoons	33	54 17-273	64 8-891	2.6×10^4 $7 \times 10^2 - 1.5 \times 10^5$
2 Cell Lagoons	28	48 3-133	105 8-891	1.2×10^5 $2 \times 10^2 - 8.5 \times 10^5$
3 Cell Lagoons	9	22 9-40	31 7-54	2.7×10^4 $2 \times 10^2 - 2.7 \times 10^4$
Activated Sludge	46	40 8-180	72 5-162	2.2×10^5 $5.2 \times 10^3 - 2.8 \times 10^5$
Trickling Filter	178	37 20-71	80 44-149	7.6×10^5 $1.7 \times 10^5 - 1.6 \times 10^6$
Primary	40	165 84-315	131 32-182	4.4×10^6 5.1×10^3 8.7×10^6

2. Determination of the relative performance of the first, second, and third cell of a three-cell lagoon.

3. Determination of the quality of a sand filtered lagoon effluent and the feasibility of slow sand filtration.

In order to meet the study objectives and take into account seasonal variations, a minimum study period of one year was required. Field sampling was planned quarterly during 30-day periods in winter, spring, summer, and fall. The first two sampling periods are complete and the third is currently in progress. The fourth sampling period is scheduled in October and November, 1974.

The sampling scheme selected was based on the availability of laboratory support and the minimum number of stations that would meet the study objectives. The eight sampling stations used were the raw waste at the influent wet well, the effluent from the first, second, and third cells, and effluent from four pilot plant sized sand filters, two each, following the second and third cells.

Automatic compositors were used at each station to collect 24-hour composite samples for all analyses except biological. These samples were analyzed under the scheme shown in Table 2.

In addition, daily flow measurements were taken from the influent totalizer and from the continuous recording of head measurements on the Cipolletti Weir in the third cell effluent channel. Two types of sand filters were used at each location. The first filter utilized a prepared sand with an effective size of 0.3 mm and a uniformity coefficient of 2.5, while the second filter was filled with river-run sand with an effective size and uniformity coefficient of 0.22 mm and 3.5, respectively. After the winter study, all sand filters were converted to prepared sand because the river-run sand filters plugged throughout their entire depth necessitating complete replacement. The sand filters were constructed out of barrels welded together and had a depth of 1.07 meters (42 in) of sand over 0.30 meters (12 in) of coarse gravel underdrain. They were considered plugged when the head loss through the filter exceeded 1.22 meters (4 ft).

Results

Table 3 presents a summary of analytical results for the winter sampling period. The influent data are divided in two sections with significantly different results. The reason for the discrepancy is due to a change in type of automatic compositor used on the influent. Initially a peristaltic pump driven compositor was used. Equipment failure necessitated a change and a vacuum activated compositor was substituted. The vacuum operation results in a collection of a greater proportion of solids and also affects the concentration of other parameters which can be present in either dissolved or particulate form. The variation between compositors has been documented by Harris and Keffer (1974). For evaluation between cells and overall performance, the data collected between February 3 to 20, 1974, should be utilized.

During the winter period 84 percent of the BOD_5 removal occurred in Cell 1. The second and third cells provided essentially no additional BOD_5 removal, but were effective in reducing concentrations of COD and NFS by 32 and 23 percent, respectively. The influent fecal coliform count was decreased from 475,000 to 22,000 organisms/100 ml

Table 2. Laboratory analyses, Blue Springs Lagoon Study.

Daily 24 Hour Composite	24 Hour Composite Taken 7 of the 30 Day Period	Miscellaneous
5 Day BOD @ 20°C	(A) *Non-filtered*	(A) *Daily*
COD	2, 5, 10, 20, 35, 56 Day BOD	Total Coliform
TOC	Nickel	Fecal Coliform
Non-filterable Solids	Lead	(B) *Weekly*
Volatile Solids	Cadmium	Phytoplankton enumeration and identification
Total Phosphorus	Zinc	Chlorophyll *a*
Ammonia Nitrogen	Copper	
Nitrite-Nitrate Nitrogen	Chromium	
Total Kjeldahl Nitrogen	(B) *Filtered (0.45 μ)*	
Turbidity	5 Day BOD	
Alkalinity	COD	
pH	TOC	
Specific Conductance	Total Volatile Solids	
	Total Phosphorus	
	Ammonia Nitrogen	
	Total Kjeldahl Nitrogen	

Table 3. Summary of operating data, Blue Springs lagoon system, average values, January 22 to February 20, 1974.

Sampling Station	Flow mgd	Temp. °C	pH	BOD_5 mg/l	COD mg/l	NFS mg/l	Tot P mg/l	NH_3-N mg/l	NO_2-NO_3-N mg/l	TKN mg/l	Tot N mg/l	Fecal Coliform Organ/ 100 ml
Influent[a] Wet Well 1/22-2/2/74	3.0	12.7	7.57	42.5	171	63	3.95	3.53	1.71	6.91	8.62	420×10^3
Influent Wet Well 2/3-2/20/74	1.48	11.6	6.35	176	900	307	8.75	10.67	0.12	15.94	16.06	475×10^3
Effluent Cell 1		4.5	7.39	28.1	119	17	6.66	10.25	0.10	12.63	12.73	69.7×10^3
Effluent Cell 2		4.1	7.34	27.9	89	13	6.58	10.63	0.10	12.57	12.67	29.9×10^3
Effluent Cell 3	2.46	3.7	7.27	26.8	81	13	6.93	11.05	0.10	12.86	12.96	22×10^3

[a] Automatic compositor changed on 2/02/74 from ISCO 1391 to QCEC CVE.
ISCO operation is by peristaltic pump, CVE by vacuum. Experience to date shows vacuum operation collects more solids.

in the final effluent. The data demonstrates the ability of a three cell system to meet secondary treatment limits for BOD_5 and NFS, but shows the difficulty in meeting fecal coliform requirements without disinfection.

The April and May, 1974 data are summarized in Table 4. With increasing water temperatures, the overall system efficiency for BOD_5 removal was 88 percent. The final effluent concentration of BOD_5 was 23.2 mg/l, well below the 30 mg/l secondary treatment limit. The concentration of NFS (Table 4) also met secondary treatment criteria. The fecal coliform count in the final effluent (Table 4) was still in excess of prescribed limits.

Data from winter and spring sand filter operation are presented in Tables 5 and 6. The winter operation was conducted in a heated shelter which may have affected the results. Between the winter and spring sampling periods, the two filters containing river-run sand were changed to prepared sand. Until the summer and fall data are complete, no attempt will be made to correlate application rates and filter efficiency.

Phytoplankton and chlorophyll *a* data for winter (January), Spring (April), and Summer (August) sampling periods are summarized in Tables 7 through 9.

Table 4. Summary of operating data, Blue Springs lagoon system, average values, April 22 to May 24, 1974.

Sampling Station	Flow mgd	Temp. °C	pH	BOD_5 mg/l	COD mg/l	NFS mg/l	Tot P mg/l	NH_3-N mg/l	NO_2-NO_3-N mg/l	TKN mg/l	Tot N mg/l	Fecal Coliform Organ/ 100 ml
Influent Wet Well	1.76	13.3	7.14	196	621	557	8.93	11.03	0.41	19.25	19.66	534×10^3
Effluent Cell 1		18.2	7.51	61.2	208	106	8.44	6.78	0.07	16.77	16.84	14.1×10^3
Effluent Cell 2		18.7	7.55	30.9	124	43	8.59	7.30	0.06	13.00	13.06	7.5×10^3
Effluent Cell 3	1.83	20.1	7.62	23.2	94	26	8.70	8.23	0.05	12.45	12.50	3.8×10^3

Table 5. Summary of operating data, Blue Springs lagoon system, Cell 2 and Cell 3 sand filters, average values, January 22 to February 20, 1974.

Sampling Station	Flow gpm[a]	Temp. °C	pH	BOD_5 mg/l	COD mg/l	NFS mg/l	Tot P mg/l	NH_3-N mg/l	NO_2-NO_3-N mg/l	TKN mg/l	Tot N mg/l	Fecal Coliform Organ/ 100 ml
Prep. Sand Cell 2, Filt. A	0.50		7.26	23.2	69	10	6.49	11.00	0.10	12.82	12.92	26×10^3
River-run Sand Cell 2, Filt. B	0.50		7.30	20.3	61	9	6.50	10.76	0.10	12.40	12.50	21×10^3
Prep. Sand Cell 3, Filt. C	0.50		7.29	25.0	82	9	6.69	11.05	0.10	12.90	13.00	19×10^3
River-run Sand Cell 3, Filt. D	0.50		7.32	19.8	62	10	6.67	11.01	0.10	12.73	12.83	15×10^3

[a] Flow applied at rate of 10 mgad–no plugging.

The winter season phytoplankton population was less than 5,000 cells/ml in each of the three cells. Correspondingly, the concentration of chlorophyll *a* in each of the cells was less than one microgram per cubic meter ($\mu g/m^3$). The predominate algae forms during this period were *Oscillatoria*, a filamentous blue-green, and *Chlamydomonas*, a green flagellate.

Filtering the effluent from Cell 2 significantly altered the phytoplankton community structure. *Chlamydomonas*, which represented only 28 percent of the total count from Cell 2, had little trouble passing through the sand filter and represented 56 percent of the total count from Filter A effluent. *Oscillatoria*, the filamentous alga, and a notorious

Table 6. Summary of operating data, Blue Springs lagoon system, Cell 2 and Cell 3 sand filters, average values, April 22 to May 24, 1974.

Sampling Station	Flow mgd	Temp. °C	pH	BOD_5 mg/l	COD mg/l	NFS mg/l	Tot P mg/l	NH_3-N mg/l	NO_2-NO_3-N mg/l	TKN mg/l	Tot N mg/l	Fecal Coliform Organ/ 100 ml
Prep. Sand Cell 2, Filt. A	a		7.45	20.8	75	13	8.38	9.41	0.09	12.53	12.62	670
Prep. Sand Cell 2, Filt. B	b		7.47	18.5	74	12	8.14	8.14	0.04	12.05	12.09	680
Prep. Sand Cell 3, Filt. C	c		7.50	15.0	66	11	8.82	9.57	0.05	12.46	12.51	220
Prep. Sand Cell 3, Filt. D	d		7.56	14.2	66	11	8.28	9.01	0.04	11.86	11.90	210

[a] 4/21 - 5/2 10 mgad; 5/2 - 5/14 1.25 mgad; 5/15 - 5/24 1.25 mgad.
[b] 4/21 - 5/4 5 mgad; 5/5 - 5/14 2.5 mgad; 5/15 - 5/24 2.5 mgad.
[c] 4/21 - 4/24 10 mgad; 4/24 - 4/29 10 mgad; 4/30 - 5/24 1.25 mgad.
[d] 4/21 - 4/28 5 mgad; 4/29 - 5/12 5 mgad; 5/13 - 5/24 2.5 mgad.

Table 7. Blue Springs Lagoon Study, winter 1974.

Cell 1	Cell 2	Filter A	Cell 3	Filter C
Phytoplankton Count (cells/ml)				
2,464	2,464	1,232	3,157	1,848
Predominate Algae (% of total)				
Oscillatoria 59% Chlamydomonas 28%	Oscillatoria 47% Chlamydomonas 28%	Chlamydomonas 56% Oscillatoria 37%	Oscillatoria 63% Chlamydomonas 24%	Chlamydomonas 46% Oscillatoria 29%
Chlorophyll a (micrograms/m^3)				
0.576	0.630	0.683	0.790	0.523

filter-clogging form, was passed through the filter, but in broken filaments and in a much smaller quantity. Similar results were observed in Cell 3 and Filter C effluent, even though the phytoplankton cell counts were reduced by as much as 50 percent. By sand filtration, the chlorophyll *a* data did not reflect this reduction because the concentrations were bordering on the lower detection limit.

By April, warmer water temperatures and increasing daylight hours promoted heavy phytoplankton growth. Phytoplankton cell counts approaching or exceeding 100,000 per milliliter were common in all three lagoons through April and continuing into the mid-part of May. The predominate algal form during this period was the coccoid green *Micractinium.* By filtering the effluent from Cell 2, a 95 percent reduction in phytoplankton cells, and a 42 percent reduction in chlorophyll *a* content was accomplished.

The filtration fragmented the *Micractinium* which allowed some passage through the filter of broken cells and dislocated setae. A small

Table 8. Blue Springs Lagoon Study, spring 1974.

Cell 1	Cell 2		Cell 3	
Phytoplankton Count (cell/ml)				
68,684	116,424		96,096	
Predominate Algae (% of total)				
Micractinium 97% Oscillatoria 1%	Micractinium 73% Oscillatoria 21%		Micractinium 96% Euglena 1%	
Chlorophyll a (micrograms/m^3)				
24.03	24.03		16.02	
		Filter A		Filter C
		5,852		7,392
		Micractinium (fragments) 35% Euglena 22%		Euglena 42% Micractinium (fragments) 16%
		13.35		5.07

Table 9. Blue Springs Lagoon Study, summer 1974.

Cell 1	Cell 2		Cell 3	
Phytoplankton Count (cell/ml)				
225,302	142,758		108,262	
Predominate Algae (% of total)				
Oscillatoria 93% Chlamydomonas 2%	Oscillatoria 78% Euglena 6%		Oscillatoria 87% Euglena 2%	
Chlorophyll a (micrograms/m^3)				
2,242.80	883.77		333.75	
		Filter A		Filter C
		9,086		11,858
		Oscillatoria 47% Ankistrodesmus 25%		Oscillatoria 61% Euglena 16%
		14.68		12.01

flagellate, *Euglena*, also passed through the filter without much difficulty.

Filtering the effluent from Cell 3 also achieved similar reductions in phytoplankton and chlorophyll *a*. The phytoplankton cell counts declined 77 percent with a corresponding 32 percent decline in chlorophyll *a* concentrations in Filter C effluent.

In mid-summer, all three lagoons supported a dense blue-green algae population dominated by *Oscillatoria*. Phytoplankton cell counts were well over 100,000 cells/ml and chlorophyll *a* ranged from 2,242 to 333 $\mu g/m^3$. As of this writing, biological monitoring has indicated the phytoplankton cell counts exceeding 500,000 ml and remedial copper sulfate treatment will begin the week of August 19. Nevertheless, filtration of Cell 2 and 3 effluents demonstrates the effectiveness of sand filtration and the subsequent reduction of algae cells, and chlorophyll *a* content.

Dye studies were conducted on June 3, 1974, to measure flow-through time. All three cells were dosed simultaneously with rhodamine WT dye solution. Break through on Cell 1, 16.8 hectares (41.5 acres), occurred in 5 hours 10 minutes. Cell 2 which is 5.0 hectares (12.4 acres) had initial break-through in 4 hours 45 minutes. The first break-through in Cell 3, 1.7 hectares (4.15 acres), occurred 45 minutes after release.

Our Blue Springs lagoon study is about half finished. The summer intensive data which are being generated presently, must be compiled and the copper sulfate dosing and fall intensive are yet to be done. The progress to date, as summarized, does not include consideration of all the analytical data but only those related to the major objectives. We are hopeful that the total results will provide additional insight into design and operation of three-cell systems.

Reference

Harris, D. J., and William J. Keffer. 1974. Wastewater sampling methodologies and flow measurement techniques. EPA 907/9-74-005.

COST-EFFECTIVENESS ANALYSIS FOR WATER POLLUTION CONTROL

R. Smith[1]

Over the past 10 years, the direction of the federal program in water pollution control has shifted from the purely scientific aspects of the problem to the more practical considerations of design and cost-effectiveness (EPA, 1971). One of the first indications of this change in attitude appeared in EPA regulations published in the Federal Register on July 2, 1970. Subparagraph 601.36 entitled *Design* reads as follows:

> No grant shall be made for any project unless the Commissioner determines that the proposed treatment works are designed so as to achieve economy, efficiency, and effectiveness in the prevention or abatement of pollution or enhancement of the quality of the water into which such treatment works will discharge and meet such requirements as the Commissioner may publish from time to time concerning treatment works design so as to achieve efficiency, economy, and effectiveness in waste treatment.

The Federal Water Pollution Control Act Amendments of 1972 (PL 92-500) further charged EPA with a number of specific tasks relating to development and publication of information and guidelines.

Two principal milestones were mentioned in the law. The first is achievement of secondary treatment effluent standards in all publicly owned treatment works by July 1, 1977, and the second is achievement of best practicable treatment in all publicly owned treatment works by July 1, 1983.

The definition of secondary treatment was published in the Federal Register on August 17, 1973. Principal provisions of this definition were that the mean value of effluent samples collected over 30 consecutive days must not exceed 30 mg/l for 5-day BOD, 30 mg/l for suspended solids, 200/100 ml for fecal coliform bacteria, and the pH of the effluent must fall between 6.0 and 9.0.

Although the formal definition of best practicable treatment has not, as yet, been published, it is expected that the definition will not contain a minimum effluent quality. The term "best practicable treatment" seems to be closely related to the concept of cost-effectiveness. The term best practicable treatment is first mentioned in PL 92-500 in Section 201 (g)(2)(A) which provides that the grant application must satisfactorily demonstrate to the Administrator that:

> ...alternative waste management techniques have been studied and evaluated and the works proposed for grant assistance will provide for the application of the best practicable waste treatment technology over the life of the works consistent with the purposes of this title.

The term "best practicable technology" is defined in PL 92-500 with reference only to non-publicly owned facilities but in Section 304 (b)(1)(B) the following statement appears:

> Factors relating to the assessment of best practicable control technology ... shall include consideration of the total cost of application of technology in relation to the effluent reduction benefits to be achieved from such application, and shall also take into account the age of equipment and facilities involved, the process employed, the engineering aspects of the application on various types of control techniques, process changes, non-water quality environmental impact (including energy requirements), and such other factors as the Administrator deems appropriate.

Thus, it appears that the term best practicable treatment is used to represent the ideal situation where all aspects of each pollution control problem are properly weighted and taken into account to arrive at the ideal solution which maximizes the common good and minimizes the common cost. The law (PL 92-500) specifically states that social costs and other intangible factors are to be taken into account when the best practicable solution is determined.

While the concept of best practicable treatment is defined to include social and other intangible factors, such as conservation of energy, the concept of cost-effectiveness is more easily understood and more easily used in the solution of any specific pollution control problem. For example, Appendix A of 40 CFR, part 35, EPA regulations was published in

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the Federal Register on September 10, 1973. (See Cost Effectiveness Analysis.) These cost-effectiveness guidelines require the non-monetary aspects of the problem to be included in descriptive terms only.

Conceptually, cost-effectiveness analysis means identifying and enumerating all feasible alternative ways of achieving the required level of treatment, estimating the total cost of each alternative, and finally, with proper consideration of the non-monetary factors, selecting the least cost alternative.

The most important point made in the cost-effectiveness guidelines is that the cost of each alternative must be the total cost expressed as a present value or an equivalent continuous cash flow. Since the federal government is authorized to pay up to 75 percent of the total cost as a grant-in-aid and the states often pay another 10 to 15 percent, the share of capital cost paid by the municipality is often as little as 10 percent. On the other hand, the municipalities usually pay the entire amount for operation and maintenance.

Therefore, in making a cost-effectiveness analysis, the tendency is often to include only the monies paid by the municipality which would be the entire amount for operation and maintenance plus a small fraction of the capital cost. When this approach is taken, the least cost alternative selected will always be capital intensive. The primary purpose of the cost-effectiveness guidelines is to insist that the total cost to federal, state, and municipal sources be used as the total cost. Other important provisions of the cost-effectiveness guidelines are listed below:

1. Extent of effort used should reflect the size and importance of the project.
2. Twenty-year planning period must be used.
3. Inflation of wages and prices shall not be considered in the analysis.
4. Discount rate recommended by Water Resources Council must be used.

It is obvious that an unlimited amount of effort can be expended in seeking the most cost-effective solution to any pollution control problem. This is recognized in the cost-effectiveness guidelines by stating that the extent of the effort expended in making the analysis should reflect the size and importance of the project. This can be expressed in another way by stating that the cost of the cost-effectiveness analysis should be included as part of the project cost.

It is generally believed that design of wastewater treatment facilities tends to be capital intensive because of the relatively small share of the capital expenditure contributed by the municipality. The cost of clean water reports have consistently reported statistical data intended to show that, on the average, wastewater treatment plants in the U.S. are underloaded, reflecting overestimates for population and industrial growth in the community. Recommendations for making more precise estimates of projected population growth are given by EPA (1972).

Examples of the kinds of questions which EPA documents consistently recommend for cost-effectiveness consideration are given as follows. First, the feasibility of consolidating treatment facilities by means of gravity sewers or force mains in order to realize the economies of scale. Rough cost estimates for plants and interconnecting pipelines can often settle this question, but in cases where a real potential for cost savings exists, a more detailed and precise cost study will be necessary.

Careful consideration of the expected growth of the residential and industrial segments of the community served and the corresponding increase in load on the treatment facilities should be made. Here the potential advantages of staged construction of treatment works should be considered.

The impact on the treatment facilities of infiltration and inflow (Grants for Construction of Treatment Works, 1973; and EPA, 1974) must be considered to determine whether correction of the sewerage system is more or less costly than expansion of the treatment facilities.

The feasibility of using treated wastewater for industrial or agricultural purposes should be explored. If a demand for the treated wastewater exists, a charge imposed on the user can be used to offset the cost of treatment.

An important consideration is selection of the treatment process train which will accomplish the treatment target at a minimum cost. Treatment by land application[2] must be included in the cost-effectiveness analysis. Process reliability[3] in terms of the variability of the effluent stream quality, protection against electrical power failure, protection against flooding, and provision for taking equipment and structures out of service for periodic cleaning and maintenance should be considered. This kind of analysis cannot be made in any precise way because of the uncertain relationship between process design parameters and the effluent quality achieved. For example, the average effluent quality to be expected from an activated sludge plant treating municipal wastewater can be estimated when the character of the raw wastewater stream and the design parameters are known, but the precision of the estimate is limited. Pilot plant results are more reliable but still the precision may not be fully adequate for cost-effectiveness analysis.

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[2] See *Wastewater Treatment and Reuse by Land Application*, Vols. I and II, 1973; *Survey of Facilities Using Land Application of Wastewater*, 1973; and *Recycling Municipal Sludges and Effluents on Land*, 1973.

[3] See *Design Criterion for Mechanical, Electrical, and Fluid Systems and Component Reliability*, 1973.

Engineering judgment must play a part in selecting the set of processes needed to achieve any treatment goal. For example, if the effluent standard is 10 mg/l BOD and 15 mg/l suspended solids this is achievable with the activated sludge process but careful consideration of factors such as operating sludge retention time (SRT), hydraulic detention time, water temperature, final settler overflow rate, etc., must be provided.

If the effluent standards require that the ammonia be converted to nitrate, a similar problem exists. The engineer must carefully consider SRT and water temperature to determine if nitrification can be reliably achieved in the activated sludge process or whether a separate nitrification process is needed.

If denitrification is required by the effluent standards, a decision between dispersed floc denitrification, columnar denitrification, or the new cycling nitrification-denitrification process must be made. The ammonia removal can also be achieved by ion exchange or by breakpoint chlorination.

If phosphorus is to be removed, the required level of phosphorus in the effluent stream is a factor to be considered. Next, the designer must choose between processes such as alum or iron addition with the increased amounts of inorganic sludge produced or the Pho-Strip process which has been tried successfully at Seneca Falls, New York. Filtration is sometimes required for removal of phosphorus to very low levels or to remove additional suspended solids and BOD.

Lime clarification of raw wastewater can be used to remove phosphorus and to remove sufficient BOD to assure good nitrification in the following activated sludge process.

If a high degree of removal of organics is required, granular carbon adsorption may be needed. A package of cost and performance estimates, supplied for use with the 1975 Needs Survey, is given in Appendix A to show how the capital cost can vary with the level of treatment required.

The impact of improved methods of operation and maintenance on the effluent quality should also be given careful consideration. Some forms of instrumentation and automation are believed to have a beneficial impact on the average quality of the effluent stream as well as the variability of the quality measures. Certain kinds of automatic control, such as dissolved oxygen control in the aeration basin, are known to result in reduced operating costs. The degree of automatic control should be selected to achieve the effluent standards consistently at a minimal cost.

Finally, storm water treatment facilities should be integrated with dry weather treatment facilities so that the maximum use is made of both types of facilities in order to minimize the pollutional load on the receiving stream.

The cost-effectiveness guidelines specify that no allowance should be made for inflation in wages, power cost, or capital cost. The reason for this is that, ideally, the analysis should be made on the basis of real value rather than dollar value. Dollars are used only as a measure of real value and, therefore, when the cost of any structure, equipment item, or service is used, it must be referenced to a specific point in time. This is done by means of various kinds of indices such as the EPA sewer and treatment indices, the average hourly wages of water, steam, and sanitary system workers, wholesale price index for industrial commodities, etc. For example, a plot of the EPA indices for construction of sewers and treatment plants is shown in Figure 1. The current rate of increase for these indices is about 40 percent per year.

The justification for omitting inflation, then, is that it is assumed that the cost of all components of the facility are being inflated at the same rate. If the inflation rate for construction is known to be significantly different than the inflation rate for, say, electrical power, there is some justification for considering inflation. Another reason for not considering inflation in cost-effectiveness analysis is the uncertainty associated with estimating the rate of inflation in future years.

Perhaps the most important concept in making a cost-effectiveness analysis is the time value of money. By this, we mean that one dollar today is worth more than the promise to pay one dollar, say, one year from now. For example, one dollar today will be worth $1.05 in one year if the prevailing rate of interest is 5 percent. It is also important in

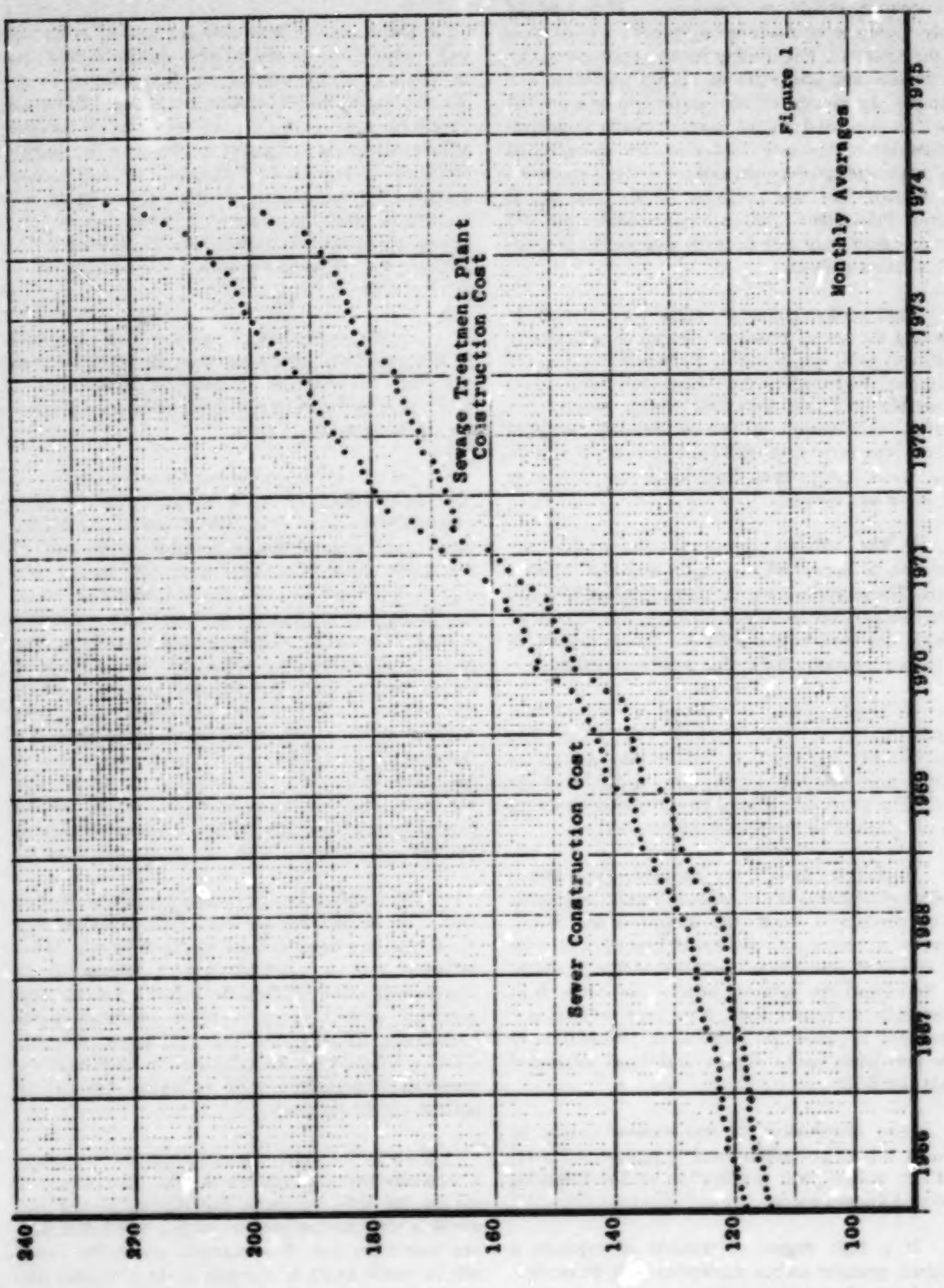


Figure 1. Sewage treatment plant construction cost index and sewer construction cost index, 1957= 100.

considering the cost of a service facility such as a treatment plant or a sewerage system to consider cost as distributed over an endless time continuum. For truly continuous costs such as the cost of wages or chemicals this presents no problem. For other items such as equipment or structures which have a finite useful life, provision must be made for replacement at the end of the useful life period. This can be done by accumulating money in a fund (sinking fund) or by borrowing the money to purchase the needed replacement item. The discarded equipment or structure may have some salvage value but this is not significant in most cases.

Thus, for cost estimation purposes, the costs incurred must be visualized as being distributed over the continuous and endless time continuum. The costs can be expressed as a continuous cash flow or as lump sums occurring at equal intervals. Since the expenditures occur in a recurring pattern, it is convenient to look at the costs over a finite interval called the planning period. The cost-effectiveness guidelines recommend the use of a 20-year planning period.

Compounding of interest is most commonly done on a yearly basis, but for illustrative purposes continuous compounding is more convenient and will be used here. The difference between continuous compounding and yearly compounding is usually small. The Water Resources Council's (1973) rate was 6 7/8 percent for fiscal year 1974 and the new recommended rate for fiscal year 1975 will be 5 7/8 percent.

When one or more equally acceptable alternatives are being studied for the purpose of selecting the least cost alternative, the stream of costs which occur over the planning period must be converted to an equivalent basis in order that the cost comparison can be made.

For example, costs for all alternatives can be converted to a present value, a future worth, or to a continuous cash flow. By present worth, we mean an equivalent lump sum expenditure which occurs at the beginning of the planning period. If we designate time from the beginning of the planning period as (t) and the length of the planning period as (T), then the present value is a lump sum payment made at $t = 0$. Similarly, future worth is defined as a lump sum expenditure made at $t = T$. By continuous cash flow, we mean a continuous stream of expenditure at a fixed rate with respect to time (R) expressed as dollars per year.

By means of simple relationships, any stream of expenditures can be converted to one of these three equivalent expressions. When the cost of all alternatives are expressed on an equivalent basis, the least cost alternative can be selected.

The actual expenditures which occur in connection with a particular alternative can be lump sums or continuous expenditures but the rate of the continuous expenditure need not be constant. If we designate a lump sum expenditure which occurs at a time (t) from the beginning of the planning period as S(t), the relationships used to convert S(t) to a present value, S(0), or to a future worth, S(T), are given below.

$$S(0) = S(t)e^{-rt} \quad \dots\dots\dots\dots \quad (1)$$

$$S(T) = S(t)e^{r(T-t)} \quad \dots\dots\dots\dots \quad (2)$$

r = discount rate, fraction

When the stream of expenditures is continuous, the expenditure at any time is expressed as Rdt but R can be a function of time. Any continuous stream of expenditures can be expressed as a present value by the following expression:

$$S(0) = \int_{t_1}^{t_2} Re^{-rt}\,dt \quad \dots\dots\dots \quad (3)$$

If R is a constant and the stream of expenditure covers the entire planning period, the present worth of the continuous cash flow is expressed as follows:

$$S(0) = R(1 - e^{-rT})/r \quad \dots\dots\dots\dots \quad (4)$$

Similarly, the continuous cash flow (R) can be expressed as a future worth as follows:

$$S(T) = R(e^{rT} - 1)/r \quad \dots\dots\dots\dots \quad (5)$$

If the continuous cash flow covers only a portion of the planning period, say from t_1 to t_2, the present value and future worth can be expressed as follows if R is a constant:

$$S(0) = R(e^{-rt_1} - e^{-rt_2})/r \quad \dots\dots\dots\dots \quad (6)$$

$$S(T) = R(e^{-rt_2} - e^{-rt_1})/r \quad \dots\dots\dots\dots \quad (7)$$

If the continuous cash flow R is expressed as a linear function such as $(a + bt)$ and covers the entire planning period, the present value of the linearly increasing stream is expressed as follows:

$$S(0) = a(1 - e^{-rT})/r + b(1 - (1 + rT)e^{-rT})/r^2 \quad . \quad (8)$$

If the continuous cash flow R increases exponentially such as $R = R_o e^{at}$, the present value of this continuous stream can be expressed as follows when the entire planning period is covered:

$$S(0) = R(e^{(a-r)T} - 1)/(a-r) \quad \ldots\ldots\ldots \quad (9)$$

Conversions between continuous cash flow, present value, and future worth are shown diagramatically in Figure 2. Six 20-year planning periods are represented in Figure 2. In the first planning period, a lump sum expenditure is shown at the 10-year point and the vertical lines on either end of the planning period represent the present value and the future worth of the lump sum expenditure made at the 10 year point. In the second planning period the future worth of a constant continuous cash flow is represented. In the third planning period the present value of a constant continuous cash flow is represented. In the fourth period the present value of a continuous cash flow between 10-15 years is shown. The cash flow in the fifth period is linearly increasing and present value is shown. The sixth period contains an exponentially increasing cash flow with the present value shown.

The normal method of financing municipal wastewater treatment plants is to issue municipal bonds and to pay the interest and principal on the bonds over a 20-30 year period as a continuous cash flow. The rate at which the money is paid can be expressed as R dollars per year, or if continuous compounding is assumed, the amount paid during the time period dt is Rdt. Here, time is expressed as years and dt is an increment of time. If the interest rate (r) is known and the time period over which the payments are to be made (T) is known, the required rate of continuous payback (R) can be calculated.

If compounding is assumed on a yearly basis, the value for R can be calculated from the following simple formula:

$$R = P^{*}I/(1 - 1/(1+I)^{**}N) \quad \ldots\ldots\ldots \quad (10)$$

R = debt service, dollars/yr
P = capital cost of plant, dollars
I = rate of interest, fraction per year
N = time period over which debt service is paid, yrs

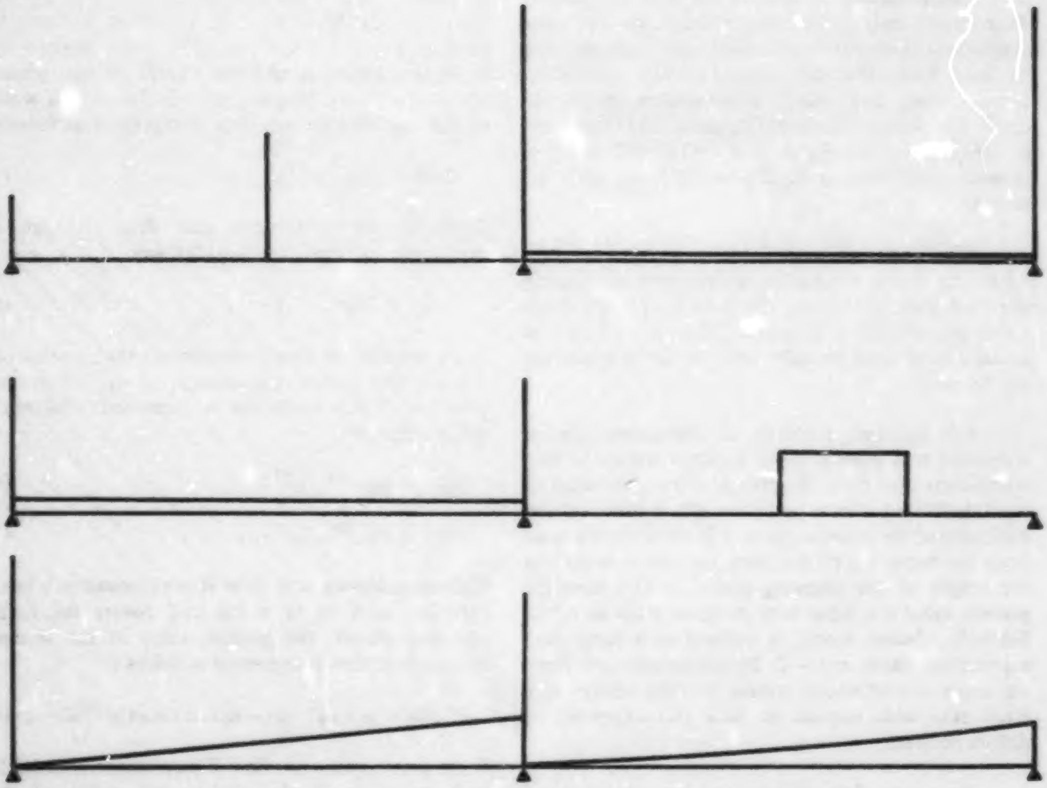


Figure 2. Diagramatic illustration of conversions between continuous cash flow, present value, and future worth 20-year planning periods, 7 percent interest.

If continuous compounding of interest is assumed, R can be calculated if we first notice that R is the sum of the interest on the unpaid balance (S) over the time period dt plus the negative derivative of the unpaid balance with respect to time. This can be seen from the diagram shown in Figure 3.

Figure 3 represents the debt service charge on a capital cost of $1,000 amortized over 20 years at an interest rate of 7 percent. The area under the curve represents the interest charges and the area above the curve represents the payments on the principal. The area above the curve between the ordinate representing zero time and any other ordinate representing a time (t) equals the initial plant cost minus the unpaid balance (S). Thus, any ordinate is the sum of the interest charges (rS) and the rate of change of (P-S) or -dS/dt.

The height of the rectangle representing the continuous cash flow can be expressed as follows:

$$R = rS - dS/dt \quad \ldots\ldots\ldots\ldots\ldots (11)$$

If we let the length of the rectangle representing the planning period be T years, Equation 11 can be integrated to find the following expression for R:

$$R = Pr/(1 - e^{-rT}) \quad \ldots\ldots\ldots\ldots\ldots (12)$$

The amortization factor, defined as the fraction of the plant cost (P) paid per year, is calculated from Equation 12 as 0.09291. If the interest is compounded yearly, the amortization factor is calculated from Equation 10 as 0.09439. The difference between these two factors is about 1.5 percent. Notice that Equation 12 is equivalent to Equation 4.

In the example on continuous compounding, the height of the rectangle is the product of the amortization factor and the plant cost taken as $1,000 or $92.91. Initially, the interest charges are $1,000 x 0.07 or $70 and the amount for repayment of the principal is $12.91. By integrating Equation 11 between any time (t) and T (20 years), the following expression for the unpaid balance is found:

$$S = R/r(1 - e^{-r(T-t)}) \quad \ldots\ldots\ldots\ldots (13)$$

Therefore, the equation of the curve in Figure 3 is just rS. Also, by differentiating Equation 13 with respect to time, we find

$$dS/dt = Re^{-r(T-t)} \quad \ldots\ldots\ldots\ldots\ldots (14)$$

Thus, it can be seen that Equation 13 can be multiplied by (r) and added to the negative of Equation 14 to equal (R).

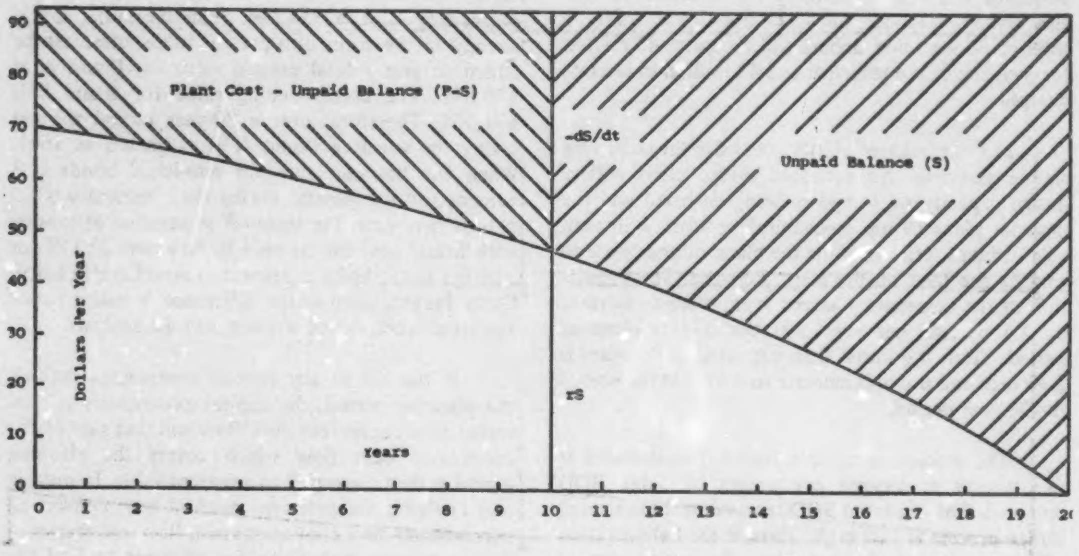


Figure 3. Distribution of continuous cash flow associated with debt service on $1000 expended at zero time amortized at 7 percent interest over 20 years.

If Equation 13 is multiplied by r and integrated between zero and T, the total amount paid in interest can be found.

$$\text{Interest Paid} = RT - R(1 - e^{-rT})/r \quad . \; . \; . \; . \; . (15)$$

In the example cited, the amount of interest paid is 85.82 percent of the plant cost and the total amount of dollars paid is 1.8582 times the plant cost.

A more general expression for the total interest charges divided by the plant cost is given below:

$$I/P = (e^{rT}(rT - 1) + 1)/(e^{rT} - 1) \quad . \; . \; . \; . \; . (16)$$

An example of the way municipal bonds are issued to provide for a payback as a continuous cash flow is shown by the notice of sale of municipal bonds for mental hygiene and retardation facilities. The notice of sale is shown in Figure 4. The expected rate of interest is approximately 6 percent compounded semi-annually.

If Equation 12 is used to estimate the continuous cash flow (R) required to pay back the $45,000,000 with interest at 6 percent, the amount which must be repaid each year is $3,475,486. The division of this amount between interest charges and principal is shown in Figure 5, assuming continuous compounding. The estimated amounts for repayment of principal from Figure 4 are shown plotted by the circled points in Figure 5.

It can be seen that the continuous compounding relationship matches the estimates reasonably well. In financing the 45 million dollar facility, a total of 86.9 million dollars will be spent, 45 million for payback of the principal and 42 million in interest charges.

For the sake of clarity, consider the following simple example. An activated sludge plant with a design capacity of 4 mgd is being designed and the designer plans to use mechanical aerators with variable effluent weirs to allow the blade submergence to vary as the load on the plant increases. The population of the community served is expected to increase by 1.5 percent per year over the 20-year planning period. Thus, the initial flow expected at the plant is 2.96 mgd and this will increase to 4 mgd at the end of the 20 year period.

The amount of oxygen required is estimated at one pound of oxygen per pound of 5-day BOD removed. The expected BOD removal in the activated sludge process is 120 mg/l. Thus, if the field aeration efficiency is 2.5 lb O_2/hp-hr, the initial rate of electrical power consumption will be 49.37 hp or 36.8 kw. The power consumption at the design capacity (4 mgd) can be estimated as 66.6 hp or 49.7 kw.

The plant designer is faced with choosing between two brands of mechanical aerators. Brand A has an initial cost of $15,000 and an average aeration efficiency of 2.5 lb O_2/hp-hr while Brand B has an initial cost of $20,000 and an average aeration efficiency of 2.7 lb O_2/hp-hr. In order to satisfy the oxygen demand at the end of the planning period, both platform mounted aerators must be rated at 75 hp.

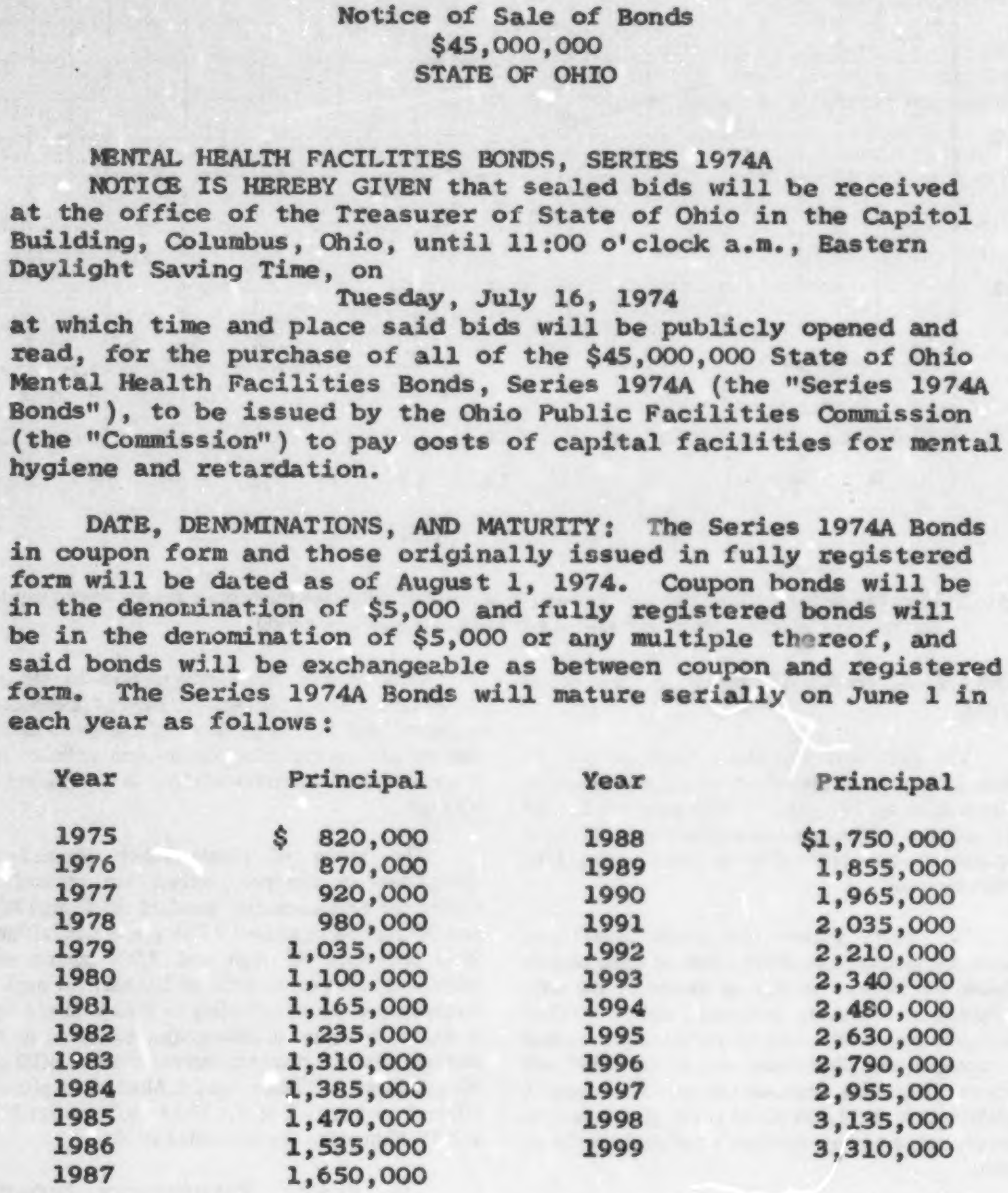
For simplicity, it will be assumed that the aeration efficiency is independent of blade submergence. If we take the cost of electrical power as 1.5 cents/kw-hr, the annual cost for electrical power to drive the mechanical aerators can be expressed as follows:

$$\$/\text{yr} = \$4{,}836\, e^{0.015t} \text{ for Brand A} \quad . \; . \; . \; . \; . (17)$$

$$\$/\text{yr} = \$4{,}477\, e^{0.015t} \text{ for Brand B} \quad . \; . \; . \; . \; . (18)$$

To convert these two streams of expenditures to a present value, we need only substitute them into Equation 3 for R. Therefore, the present value of the stream of expenditures represented by Equation 17 can be found by the use of Equation 9 in which the value for (r) is the discount rate (taken as 7 percent) minus the rate of growth (1.5 percent) or 5.5 percent. The value for (R) is $4,835.52 for Brand A and $4,477.33 for Brand B. When this is done, the present value of electrical power for Brand A aerator is computed as $58,659. For Brand B, the corresponding value is $54,304. Since the initial cost is already in the form of a present value, these can be added to give a total present value for Brand A of $73,659. The corresponding value for Brand B is $74,304. Therefore, over a 20-year period the net savings in selecting Brand A is estimated as $645. When the discount rate for municipal bonds is 7 percent and the growth rate for the community is 1.5 percent per year, the trade-off in aeration efficiency with initial cost can be seen to be about $2,178 for each 0.1 lb O_2/hp-hr increment in aeration efficiency. Other factors, such as the difference in maintenance and repair cost, can be worked into the analysis.

If the life of any process component exceeds the planning period, the capital expenditure is converted to a continuous cash flow and that part of the continuous cash flow which covers the planning period is then converted to a present value. In making cost analysis, the preferred method is to convert all expenditures to a continuous cash flow and then sum the continuous cash flows for all items to find the total cost expressed as a continuous cash flow. This cost is then often converted to a total treatment cost



Notice of Sale of Bonds
$45,000,000
STATE OF OHIO

MENTAL HEALTH FACILITIES BONDS, SERIES 1974A

NOTICE IS HEREBY GIVEN that sealed bids will be received at the office of the Treasurer of State of Ohio in the Capitol Building, Columbus, Ohio, until 11:00 o'clock a.m., Eastern Daylight Saving Time, on

Tuesday, July 16, 1974

at which time and place said bids will be publicly opened and read, for the purchase of all of the $45,000,000 State of Ohio Mental Health Facilities Bonds, Series 1974A (the "Series 1974A Bonds"), to be issued by the Ohio Public Facilities Commission (the "Commission") to pay costs of capital facilities for mental hygiene and retardation.

DATE, DENOMINATIONS, AND MATURITY: The Series 1974A Bonds in coupon form and those originally issued in fully registered form will be dated as of August 1, 1974. Coupon bonds will be in the denomination of $5,000 and fully registered bonds will be in the denomination of $5,000 or any multiple thereof, and said bonds will be exchangeable as between coupon and registered form. The Series 1974A Bonds will mature serially on June 1 in each year as follows:

Year	Principal	Year	Principal
1975	$ 820,000	1988	$1,750,000
1976	870,000	1989	1,855,000
1977	920,000	1990	1,965,000
1978	980,000	1991	2,035,000
1979	1,035,000	1992	2,210,000
1980	1,100,000	1993	2,340,000
1981	1,165,000	1994	2,480,000
1982	1,235,000	1995	2,630,000
1983	1,310,000	1996	2,790,000
1984	1,385,000	1997	2,955,000
1985	1,470,000	1998	3,135,000
1986	1,535,000	1999	3,310,000
1987	1,650,000		

Figure 4. Notice of sale of municipal bonds appearing in the Cincinnati Enquirer June 24, 1974 (in part).

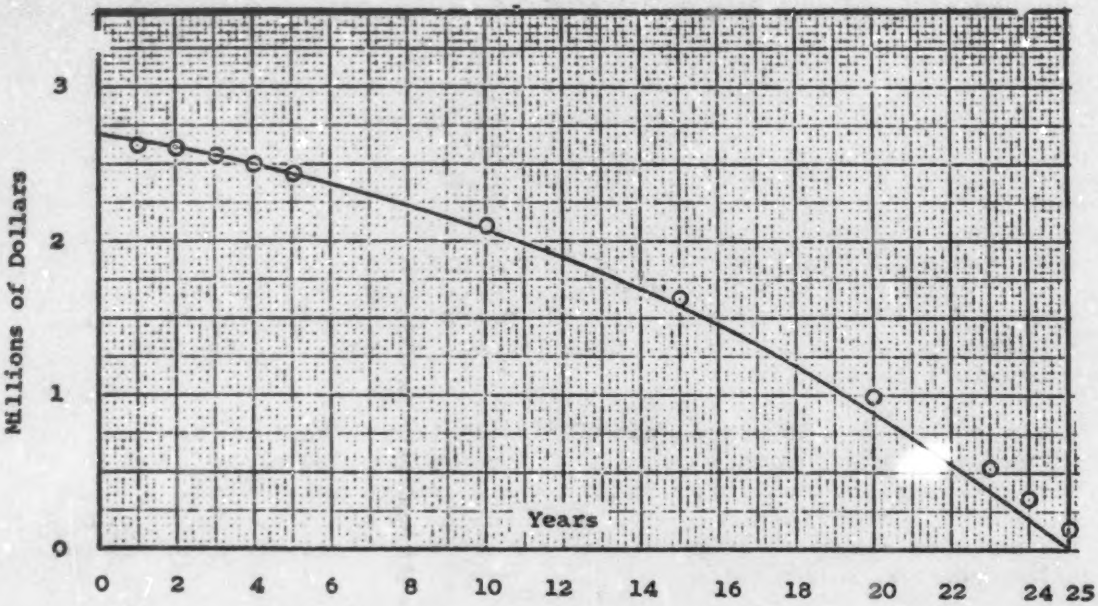


Figure 5. Distribution between interest charges and payback of principal for financing a facility with a capital cost of $45,000,000 at an interest rate of 6 percent over a 25-year period.

expressed as cents/1000 gallons of wastewater treated.

The 1973 Survey of Needs conducted by EPA asked each state to identify every plant planned for construction by 1990 and to give estimates for the cost and the effluent quality required. An analysis of the information submitted by the states produced the following results.

The survey showed that about 15,000 new plants are planned and about 6,000 of these plan to achieve an effluent quality in excess of the EPA definition of secondary treatment. About 4,000 of the 6,000 plants will make no provision for removal of nitrogen and phosphorus and about 2,000 will remove phosphorus, ammonia nitrogen or nitrogen. A breakdown of the 1,868 plants which plan to remove phosphorus, ammonia nitrogen or nitrogen is shown below.

		P Re-moval	NH_3 Re-moval	NO_3 Re-moval
Number of Plants	1,868	962	1,141	182
Total Flow, mgd	13,923	7,166	8,105	2,081
Average Flow/ Plant, mgd	7.45	7.44	7.10	11.43

Most of the plants which plan to remove phosphorus estimate the concentration of phosphorus in the effluent as 1 mg/l. Most of the plants which plan to oxidize the ammonia nitrogen estimate the concentration of ammonia nitrogen in the effluent as 1-2 mg/l.

The group of plants which planned no phosphorus or nitrogen removal but planned to exceed the EPA secondary standard of 30 mg/l BOD and 30 mg/l SS contained 3,788 plants with effluent BOD less than 30 mg/l and 3,987 plants with suspended solids in the effluent less than 30 mg/l. A breakdown of plants according to BOD level and flow is shown in Figure 6. Histograms, according to the number of plants planning various levels of BOD and SS, are shown in Figures 7 and 8. Most of the planned effluent qualities fell in the 10-15 mg/l class for BOD and 15-20 mg/l for the suspended solids.

The Federal Environmental Protection Agency's grant-in-aid program for construction of wastewater treatment represents a significant national expenditure and careful planning and design to minimize cost is easily justified.

For example, Deputy EPA Administrator John R. Quarles reported in the July 18, 1974, issue of Engineering News-Record that the total for con-

Number of Plants

BOD mg/l																		Total
26-30	303	11	5	4	0	1	2	1	0	2	0	0	1	3	1	0	0	334
21-25	553	39	18	10	8	8	3	3	0	1	0	1	0	3	2	1	1	651
16-20	1366	87	35	19	9	6	2	10	1	0	4	1	0	0	10	2	1	1553
11-15	302	34	9	2	3	2	1	4	2	0	0	1	3	1	2	4	2	372
6-10	464	40	12	8	5	3	2	1	3	1	2	0	0	1	2	1	0	545
0-5	297	16	5	8	2	2	0	1	0	1	0	1	0	0	0	0	0	333
	0-5	6-10	11-15	16-20	21-25	26-30	31-35	36-40	41-50	51-60	61-70	71-80	81-90	91-100	101-200	201-300	301-400	3788

Flow, mgd

Figure 6. Distribution of plants planning to achieve a BOD standard of less than 30 mg/l from the EPA Needs Survey for 1973.

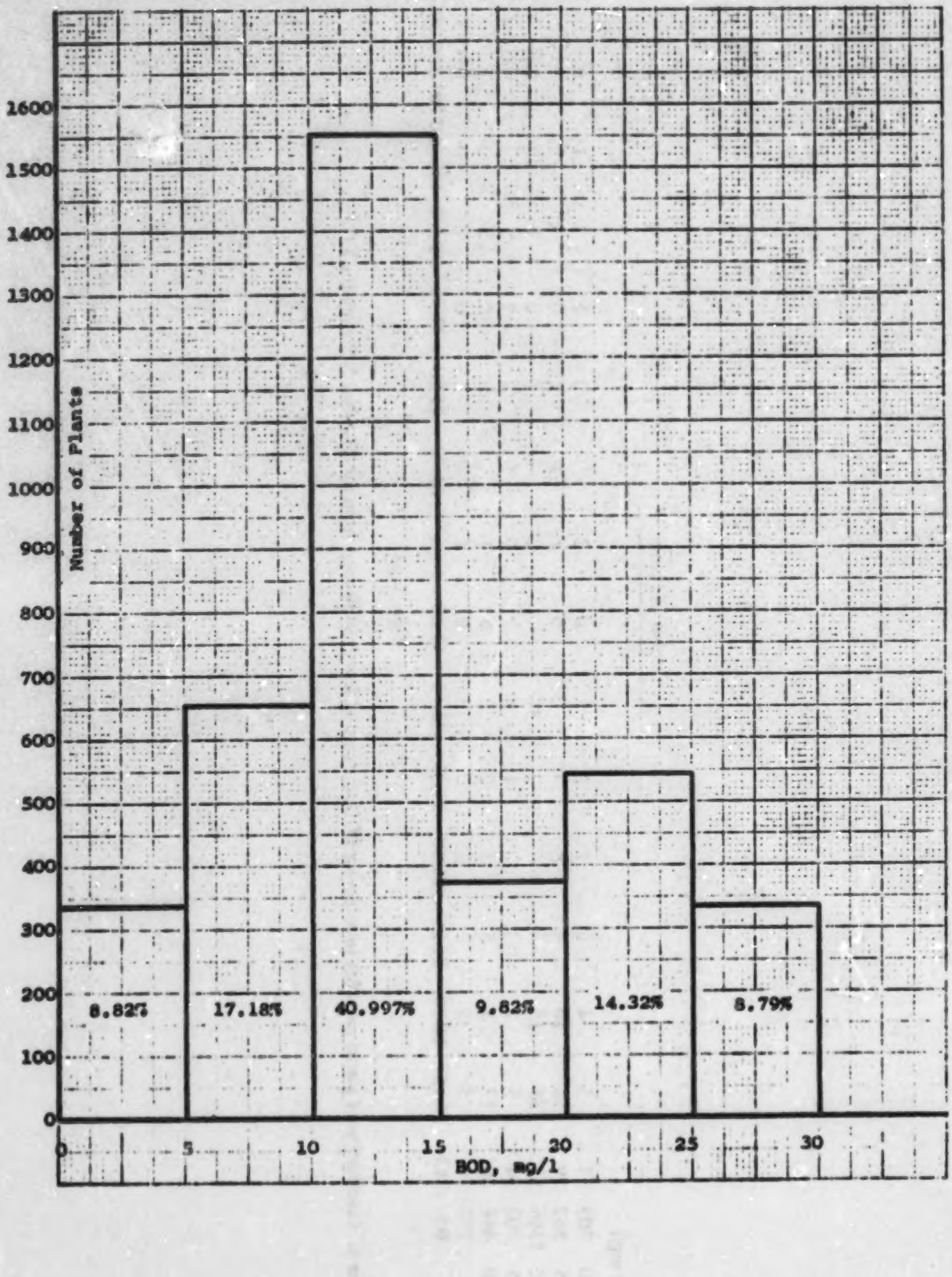


Figure 7. 1973 survey of needs municipal wastewater treatment facilities.

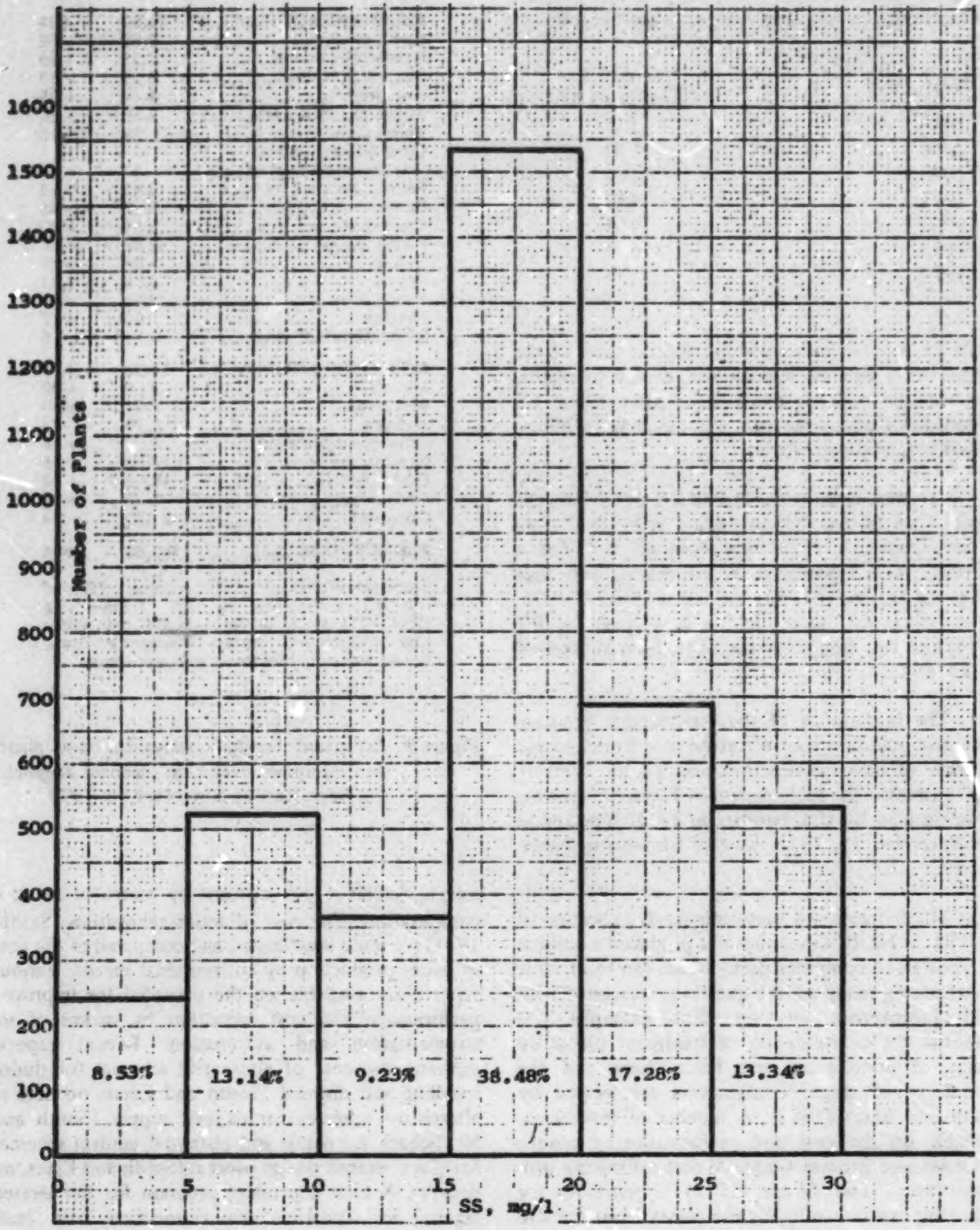


Figure 8. 1973 survey of needs municipal wastewater treatment facilities.

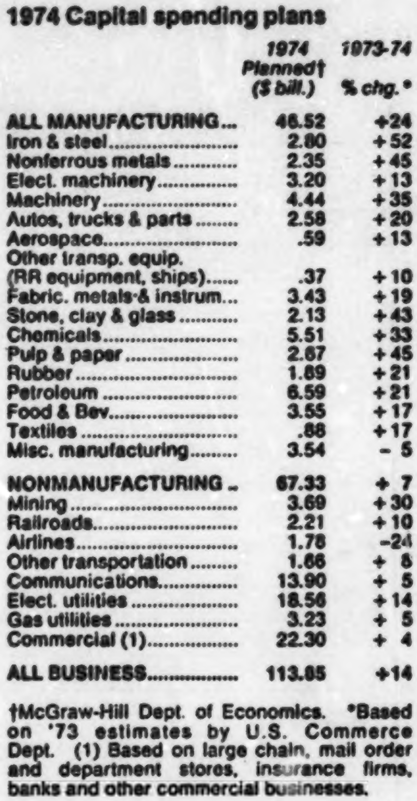
struction grant awards in fiscal year 1974 totaled about 2.6 billion dollars. The actual cash outlays, that is, the total of checks written in FY 1974 totaled about 1.5 billion dollars. According to Mr. Quarles, the cash outlays for other years have been as follows:

Year	Cash Outlay, millions of dollars
1968	120
1969	130
1970	180
1971	480
1972	410
1973	680
1974	1500

Since PL 92-500 authorizes the federal share of the construction cost of interceptors, treatment plants, and outfalls to be as much as 75 percent, the 1.5 billion dollar outlay represents as much as 2.0 billion dollars in new construction.

Congress authorized, in Public Law 92-500, the expenditure of five billion dollars in FY 1973, six billion dollars in FY 1974, and seven billion dollars in FY 1975. The Administration has released only four billion of the seven billion authorized for FY 1975, three billion of the six billion authorized for FY 1974, and two billion of the five billion authorized for FY 1973.

The importance of the construction program for water pollution control can be seen from a recent estimate of new construction planned for 1974 by U.S. business. The table shown in Figure 9, showing spending plan by U.S. business in 1974, appeared in the November 15, 1973, issue of Engineering News-Record.



1974 Capital spending plans

	1974 Planned† *($ bill.)*	*1973-74* *% chg.**
ALL MANUFACTURING...	46.52	+24
Iron & steel	2.80	+52
Nonferrous metals	2.35	+45
Elect. machinery	3.20	+13
Machinery	4.44	+35
Autos, trucks & parts	2.58	+20
Aerospace	.59	+13
Other transp. equip. (RR equipment, ships)	.37	+10
Fabric. metals & instrum.	3.43	+19
Stone, clay & glass	2.13	+43
Chemicals	5.51	+33
Pulp & paper	2.67	+45
Rubber	1.89	+21
Petroleum	6.59	+21
Food & Bev.	3.55	+17
Textiles	.88	+17
Misc. manufacturing	3.54	− 5
NONMANUFACTURING ..	67.33	+ 7
Mining	3.69	+30
Railroads	2.21	+10
Airlines	1.78	−24
Other transportation	1.66	+ 8
Communications	13.90	+ 5
Elect. utilities	18.56	+14
Gas utilities	3.23	+ 5
Commercial (1)	22.30	+ 4
ALL BUSINESS	113.85	+14

†McGraw-Hill Dept. of Economics. *Based on '73 estimates by U.S. Commerce Dept. (1) Based on large chain, mail order and department stores, insurance firms, banks and other commercial businesses.

Figure 9. Estimated capital outlays for new plants and equipment for U.S. business Engineering News Record November 15, 1973.

The Systems and Economic Analysis Section of AWTRL at NERC Cincinnati has produced a number of reports and computer decks which can be of value in examining many of the questions associated with cost-effectiveness analyses. For example, the potential for consolidation of treatment plants by means of interceptors and force mains and the advantages of staged construction are treated by Smith and Eilers (1971). A number of reports are available on the cost and performance of various processes and process trains. A cost estimating program (Eilers and Smith, 1971) is available for estimating the cost of complete plants when the size of all structures and equipment are known. An Executive Program (Eilers and Smith, 1970) is available which will solve for all process streams when performance measures are known for each process. This program is also capable of sizing all processes when the performance measures are known and also solving for all recycle streams by means of iterative computation. The cost of water renovation (Smith, 1971) for reuse is estimated and compared to the cost of water production by conventional means. Various reports are available on the potential for improved performance and cost reduction by means of instrumentation and automation. Recent reports examine the cost of alternative schemes for sludge handling and disposal (Smith and Eilers, no date a) alternative schemes for oxygen supply (Smith and McMichael, no date), and potential control schemes for the activated sludge process (Smith and Eilers, no date b). A time dependent program for the aerated lagoon was developed in connection with cost-effectiveness studies of equalization basins (Smith et al., 1973). Cost estimation programs have recently been developed for dispsered floc nitrification and denitrification, granular carbon adsorption, and aerated and facultative lagoons. For example, the program for lagoons is shown in Appendix B.

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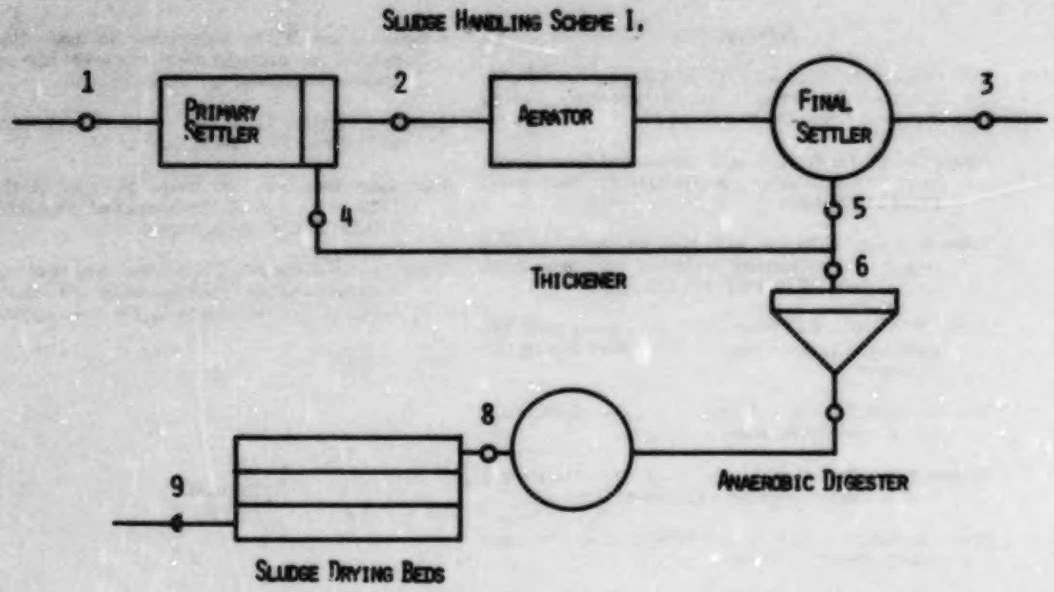
Appendix A

Table A-1. Identifying numbers for individual processes or process trains.

Primary Sedimentation I	1
Primary Sedimentation II	2
Primary Sedimentation I & Iron	3
Primary Sedimentation II & Iron	4
High Rate Trickling Filter I	5
High Rate Trickling Filter II	6
Low Rate Trickling Filter I	7
Low Rate Trickling Filter II	8
Activated Sludge I	9
Activated Sludge II	10
Activated Sludge III	11
High Rate Trickling Filter I & Alum	12
High Rate Trickling Filter II & Alum	13
Low Rate Trickling Filter I & Alum	14
Low Rate Trickling Filter II & Alum	15
Activated Sludge I & Alum	16
Activated Sludge II & Alum	17
Activated Sludge III & Alum	18
Separate Nitrification	19
Separate Denitrification	20
Filtration & Alum	21

Note: Activated sludge and trickling filter groups include primary sedimentation.


SLUDGE HANDLING SCHEME I.
1
PRIMARY SETTLER
2
AERATOR
FINAL SETTLER
3
4
5
THICKENER
6
8
ANAEROBIC DIGESTER
9
SLUDGE DRYING BEDS

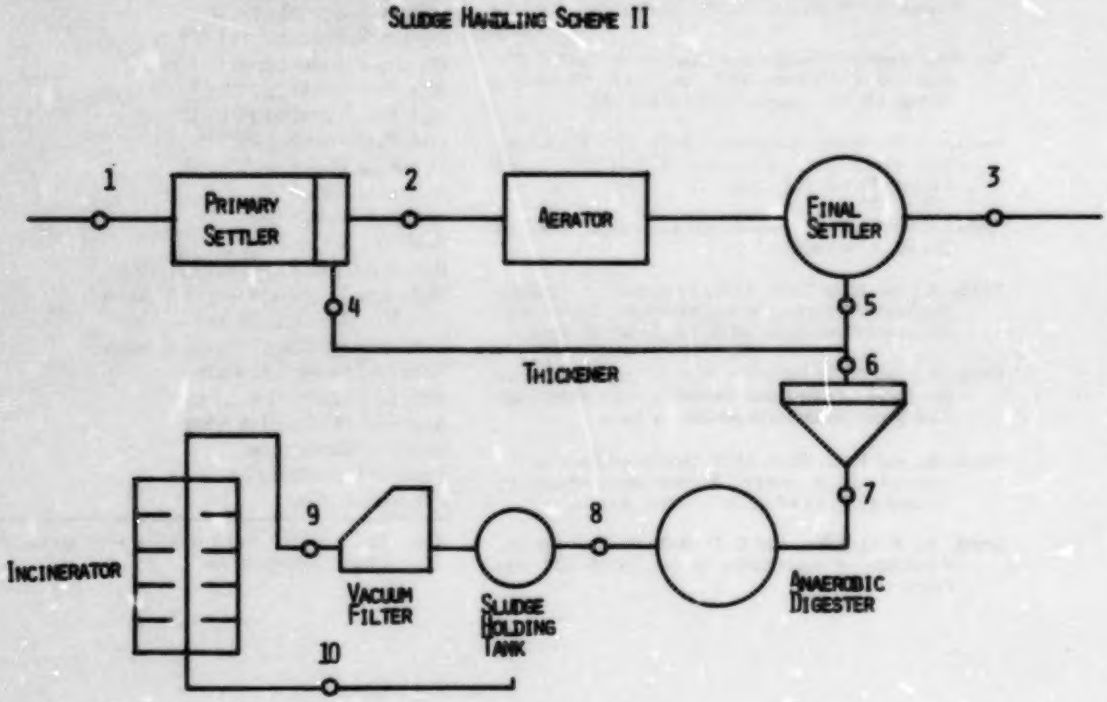

SLUDGE HANDLING SCHEME II
1
PRIMARY SETTLER
2
AERATOR
FINAL SETTLER
3
4
5
THICKENER
6
7
INCINERATOR
9
VACUUM FILTER
SLUDGE HOLDING TANK
8
ANAEROBIC DIGESTER
10

Figure A-1. Sludge handling schemes I, II, and III.

6

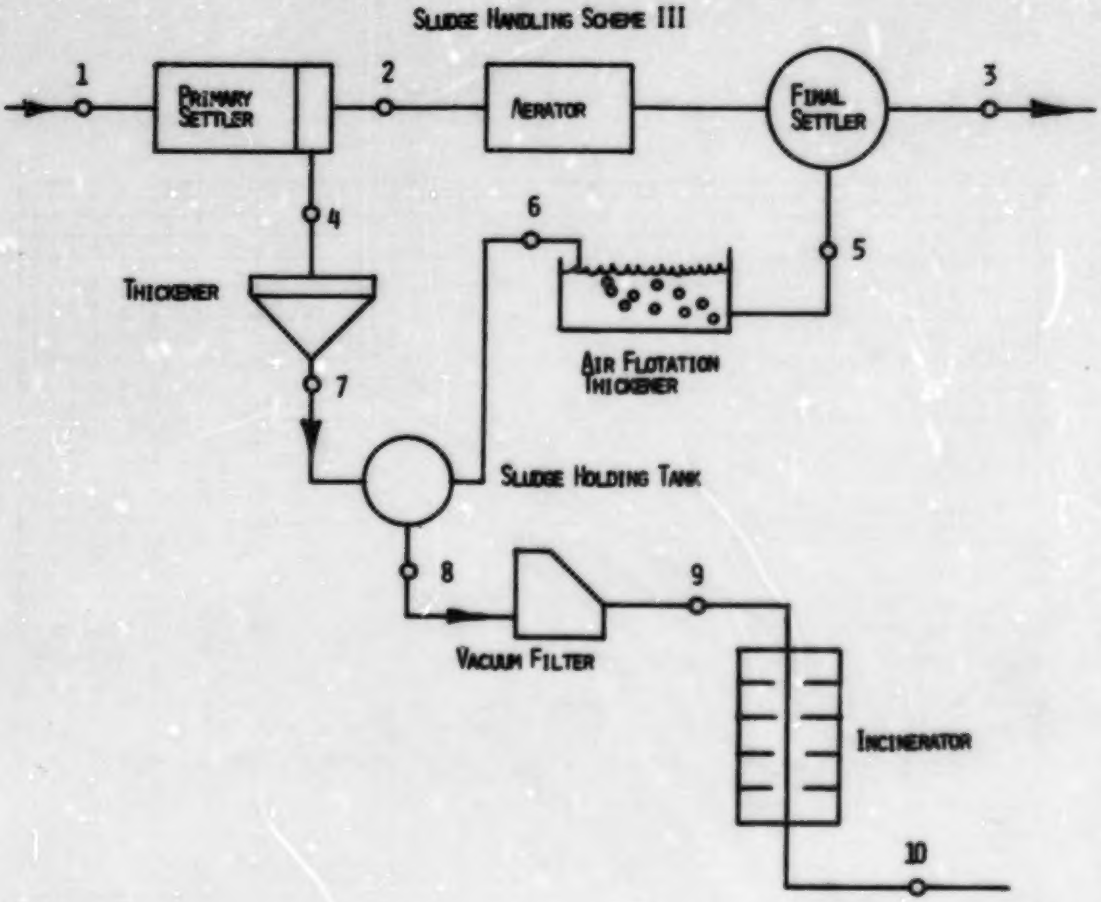

SLUDGE HANDLING SCHEME III
1
PRIMARY SETTLER
2
AERATOR
FINAL SETTLER
3
4
6
5
THICKENER
AIR FLOTATION THICKENER
7
SLUDGE HOLDING TANK
8
9
VACUUM FILTER
INCINERATOR
10

Figure A-1. Continued.

Table A-2. Estimated effluent stream quality (pollutant concentrations in mg/l) achievable with process trains identified as P1, P2, P3, and P4. 9/10/11 means activated sludge treatment with sludge handling schemes I, II, or III.

BOD	SS	COD	P	NH_3-N	NO_3-N	P1	P2	P3	P4
200	200	390	10	20	0	0	0	0	0
130	100	250	9	20	0	1/2	0	0	0
100	50	185	2	20	0	3/4	0	0	0
45	60	90	8	18	0	5/6	0	0	0
25	30	60	8	10	10	7/8	0	0	0
20	20	45	7	17	0	9/10/11	0	0	0
25	30	50	2	18	0	12/13	0	0	0
15	15	35	2	10	10	14/15	0	0	0
15	15	35	2	17	0	16/17/18	0	0	0
10	20	35	8	2	18	9/10/11	19	0	0
10	20	45	8	1	1	9/10/11	19	20	0
5	5	25	1	20	0	16/17/18	21	0	0
5	5	30	1	1	1	9/10/11	19	20	21

Note: Expected Range = ± 15 percent for all estimates.

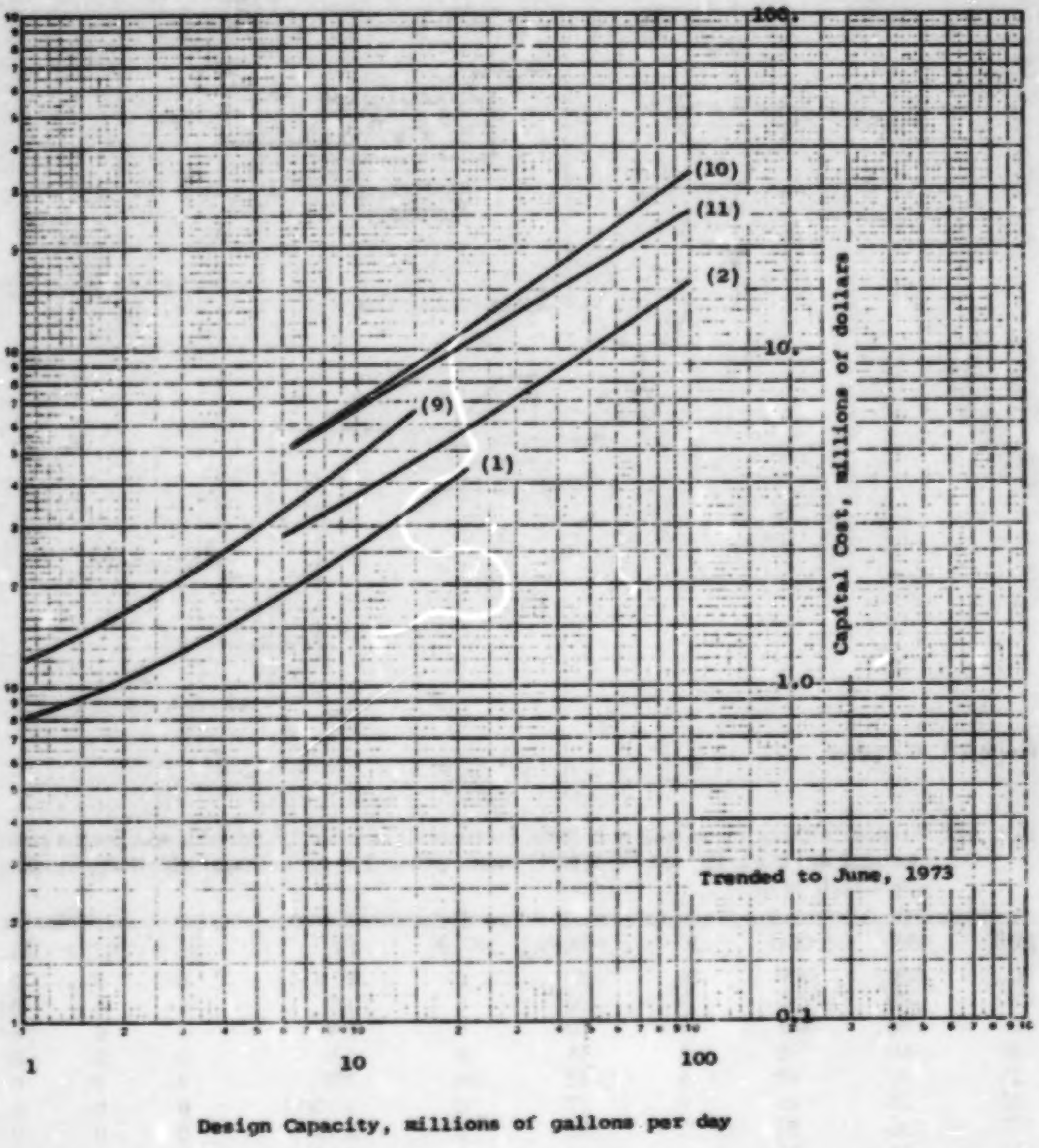


Figure A-2. Capital cost estimates for primary sedimentation plants with sludge handling schemes I (1) and II (2) and for activated sludge plants with sludge handling schemes I (9), II (10), and III (11).

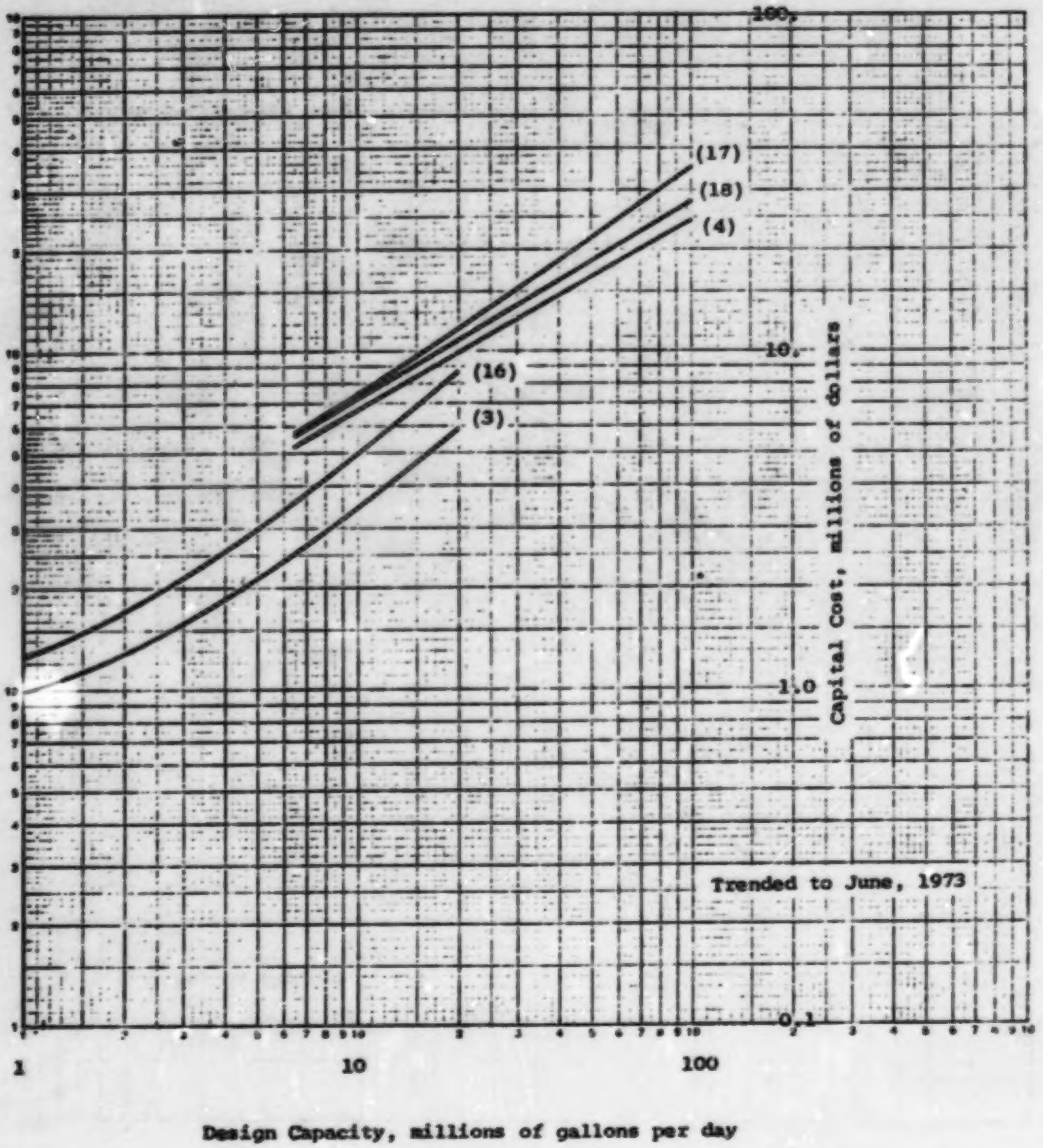


Figure A-3. Capital cost estimates for primary sedimentation plants equipped with facilities to add iron and sludge handling schemes I (3) and II (4). Capital cost estimates for activated sludge plants equipped with facilities to add alum and sludge handling schemes I (16), II (17), and III (18).

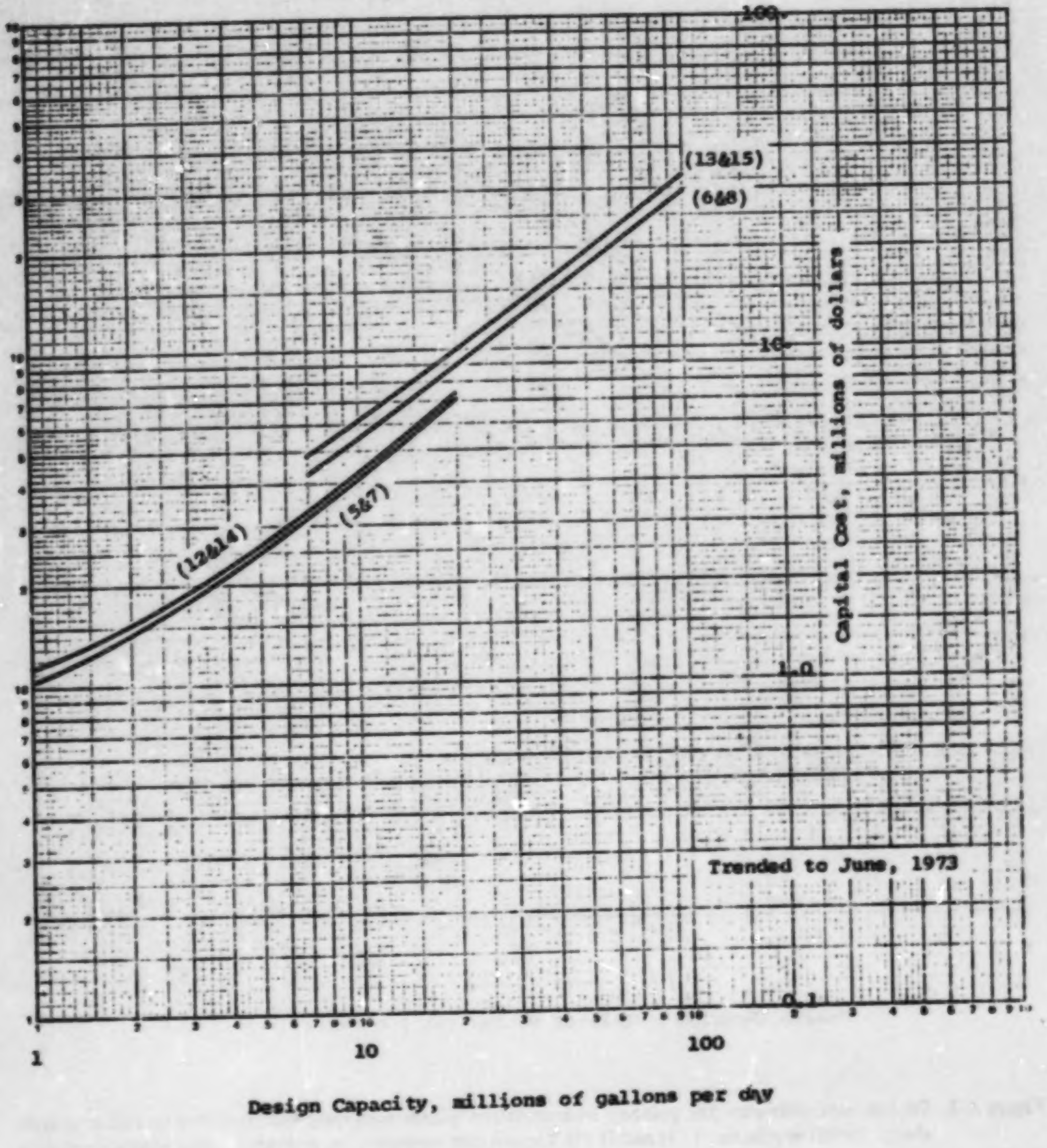


Figure A-4. Capital cost estimates for trickling filter plants with sludge handling schemes I (5 & 7) and II (6 & 8) and for trickling filter plants with facilities for adding alum ahead of the final settler with sludge handling schemes I (12 & 14) and II (13 & 15).

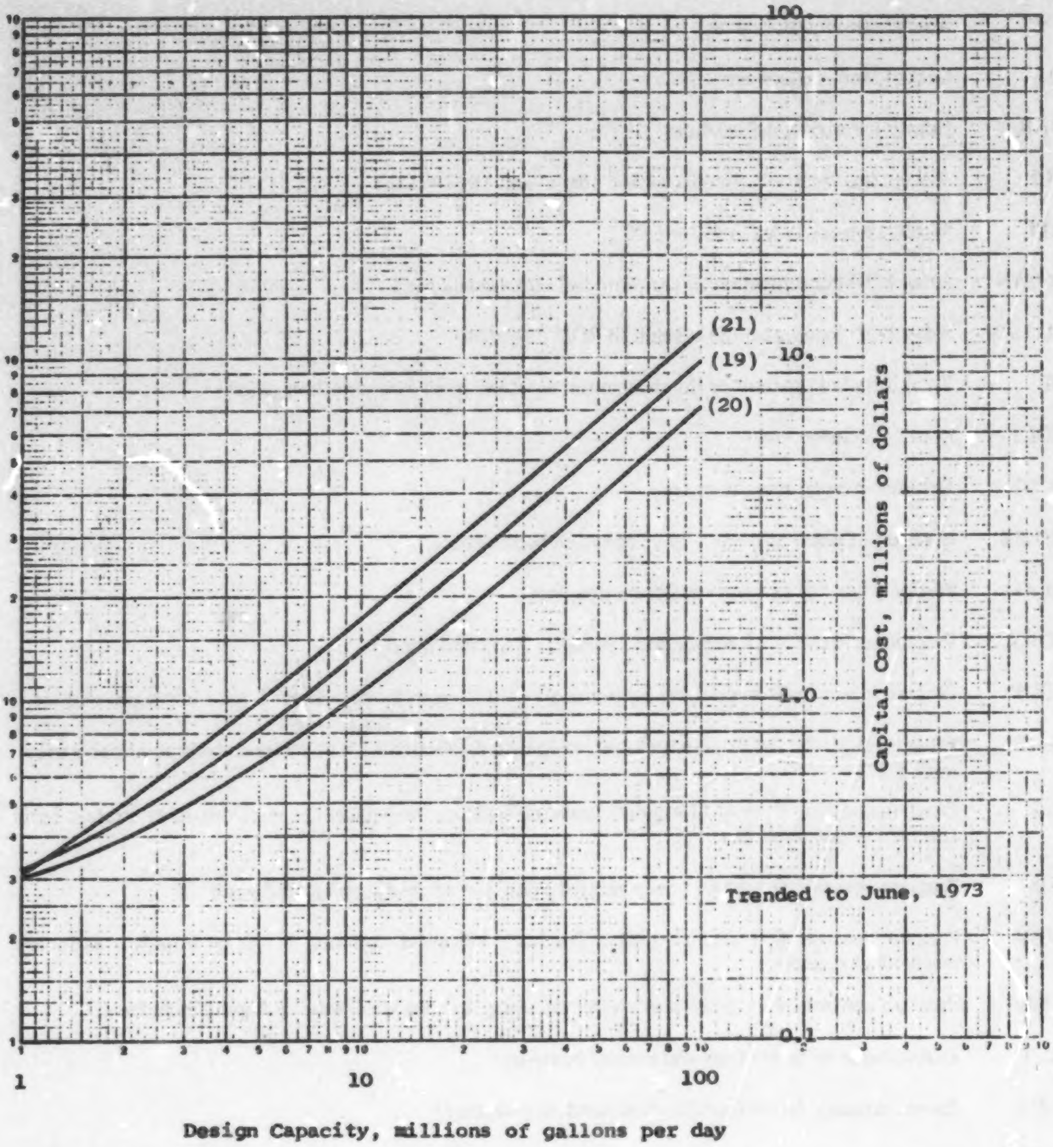


Figure A-5. Capital cost estimates for separate nitrification (19), separate denitrification (20), and filtration with supplementary alum addition (21).

Appendix B

Table B-1. Input variables to waste stabilization pond cost estimating study.

QD	Average daily volume flow used for design of the system, mgd
RI	Amortization interest rate, fraction
YRS	Amortization period, years
DA	Cost of land, dollars/acre
DHR	Hourly labor rate, dollars/hour
CCI	Sewage treatment plant construction cost index, 1967 = 1.00
WPI	Wholesale price index, 1967 = 1.00
BODIN	5-day BOD concentration of the pond influent stream, mg/l
PLOAD	5-day BOD loading on the system, lb BOD/day/acre
HP	Total installed capacity of the mechanical aerators, horsepower/million gallons
DCL2	Dose of chlorine, mg/l
TCL2	Chlorine contact time, minutes
FSLAB	Fraction of pond protected by concrete embankment
HEAD	Pumping head of raw wastewater pumps, feet
FORK	Program control: 0 = non-aerated pond, 1 = aerated pond
TEMP	Program control: 0 = non-aerated pond in a cold climate, 1 = non-aerated pond in a warm climate
SLAB	Program control: 0 = no concrete embankment protection, 1 = concrete embankment protection included
CL2	Program control: 0 = no chlorination contact basin or feed system, 1 = chlorination contact basin and feed system included
DEEP	Program control: 0 = 10 feet deep aerated pond, 1 = 15 feet deep aerated pond
SAER	Program control: 0 = small impeller floating mechanical aerators, 1 = large impeller stationary mechanical aerators
PUMP	Program control: 0 = no raw wastewater pumping, 1 = raw wastewater pumping included
ECFO	Excess capacity factor (raw wastewater pumping)
ECF1	Excess capacity factor (stabilization pond, non-aerated)
ECF2	Excess capacity factor (stabilization pond, aerated)
ECF3	Excess capacity factor (surface aerators)
ECF4	Excess capacity factor (concrete embankment)
ECF5	Excess capacity factor (chlorination contact basin)
ECF6	Excess capacity factor (chlorination feed system)

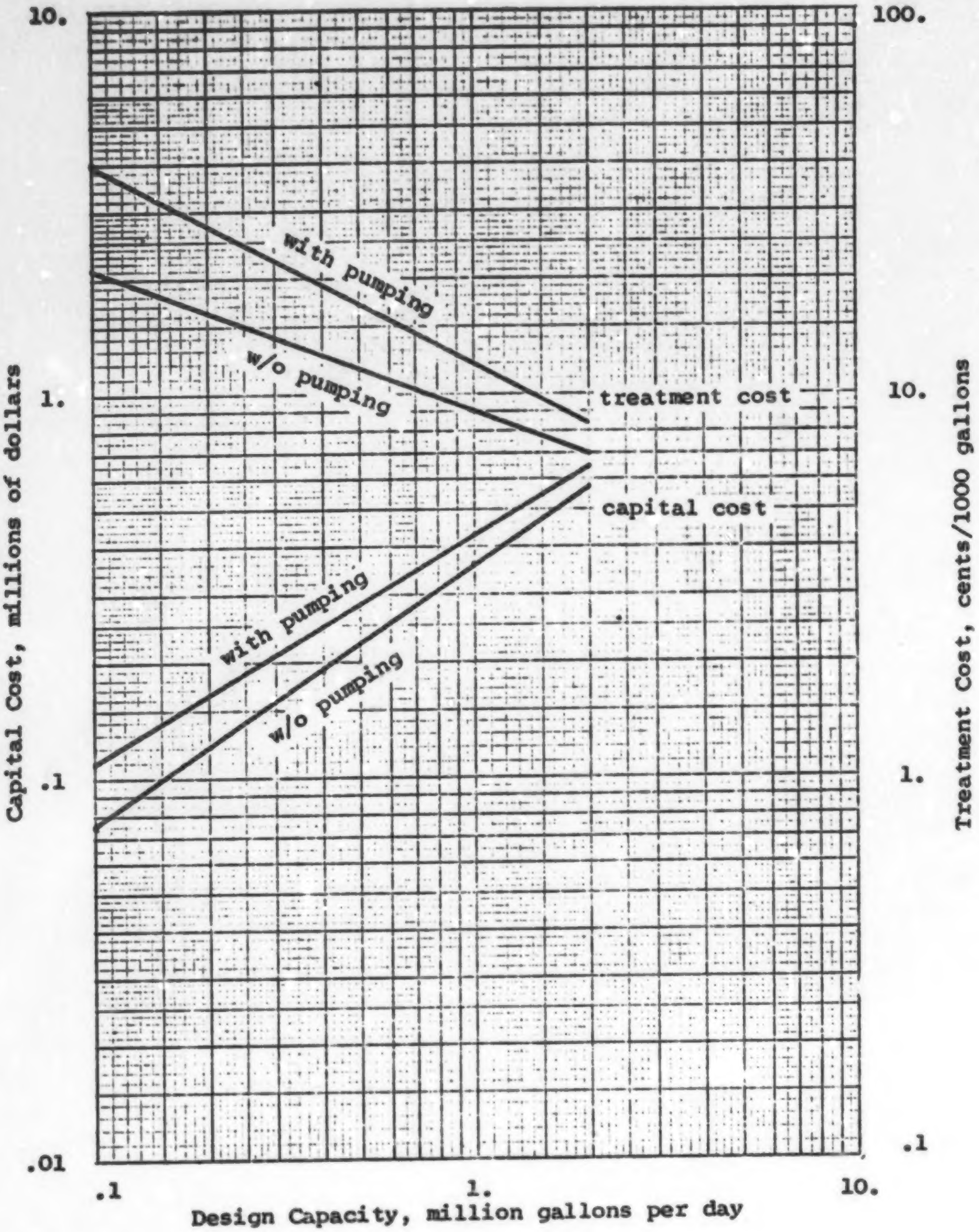


Figure B-1. Estimated cost trended to January 1974 for waste stabilization ponds in warm climates with the design criteria of 50 lbs BOD/day/acre.

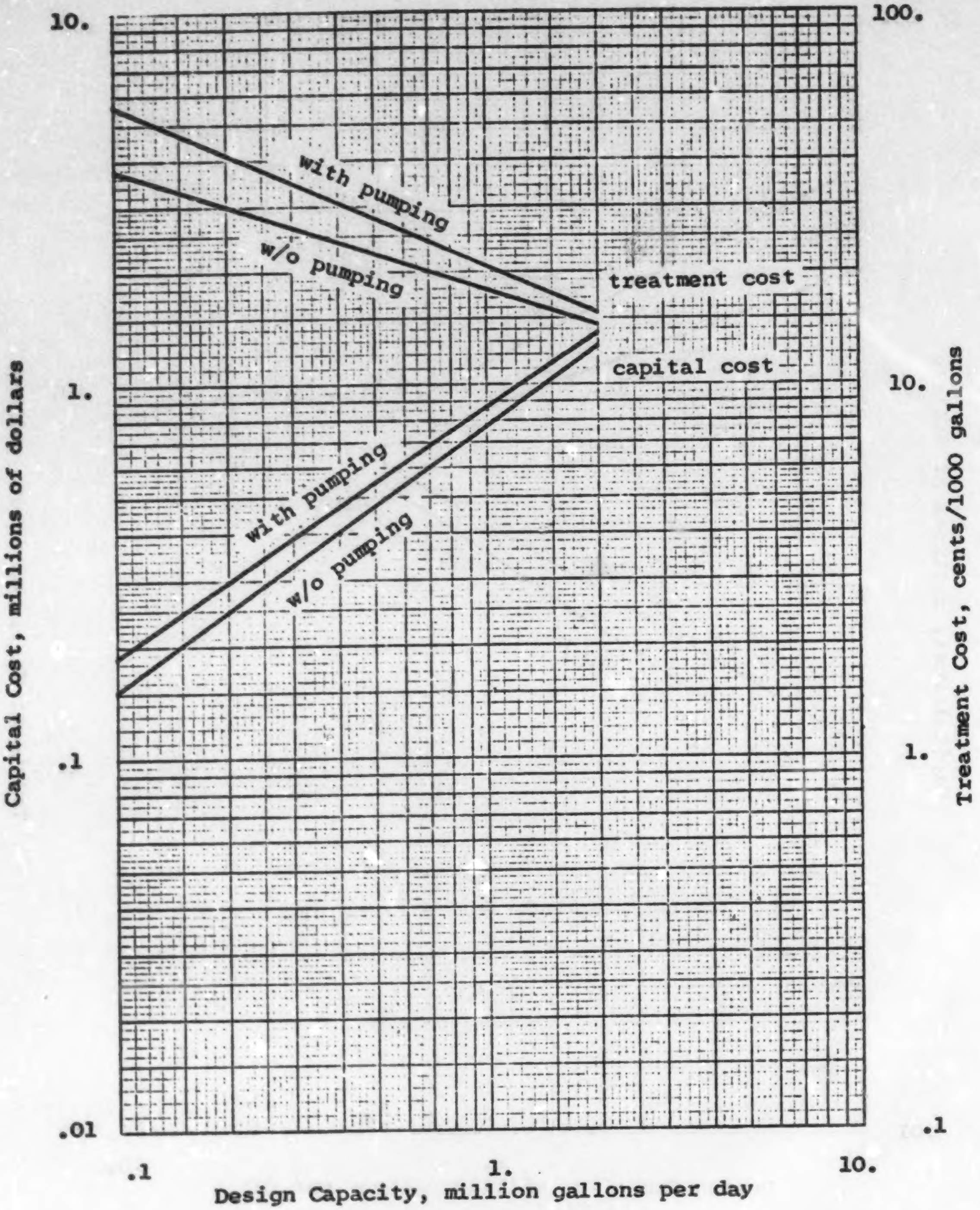


Figure B-2. **Estimated cost trended to January 1974 for waste stabilization ponds in cold climates with the design criteria of 17 lbs BOD/day/acre.**

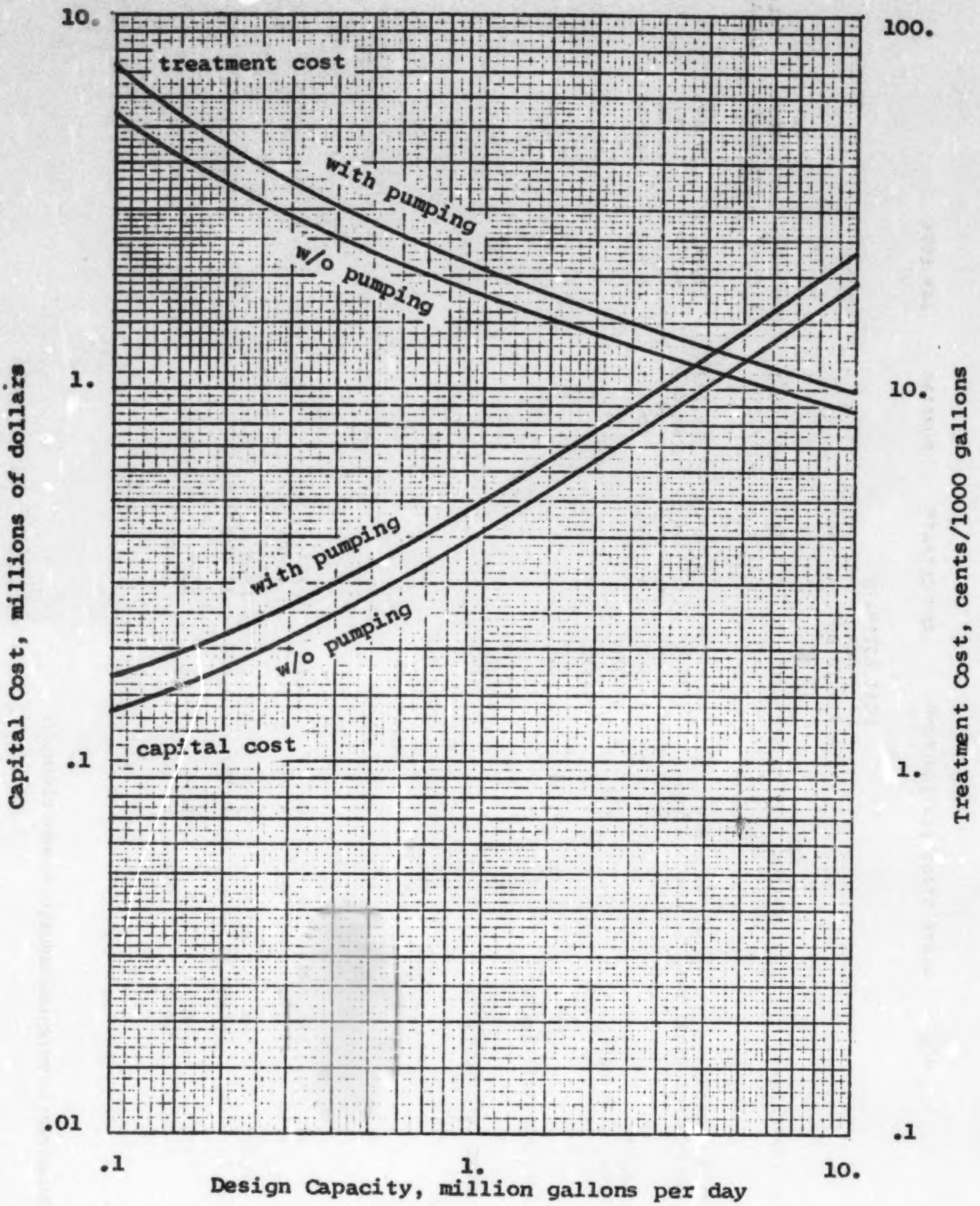


Figure B-3. Estimated cost trended to January 1974 for aerated lagoons with the design criteria of 170 lbs BOD/day/acre.

```
        .1 MGD     WASTE STABILIZATION POND     WARM CLIMATE     PUMPING     JAN 1974

                                     COST ESTIMATE
                                          FOR
                               WASTE STABILIZATION PONDS

      PROCESS COMPONENTS           DESIGN     CONSTRUCTION      CAPITAL   DEBT COST    O+M COST  TOTAL COST     EXCESS
                                 PARAMETER    COST, $           COST, $   CTS/1000     CTS/1000    CTS/1000   CAPACITY

RAW WASTEWATER PUMPING            0.10 MGD        29602.        43084.       9.233      12.262      21.496       1.00
STABILIZATION POND-NONAERATED-W   3.33 ACRES      45754.        66590.      14.271       4.720      18.992       1.00
                                              ----------    ----------  ----------  ----------  ----------
SUBTOTAL                                          75356.       109675.      23.505      16.983      40.488
ENGINEERING COST                                  14766.
LAND REQUIRED,  9.78 ACRES
LAND COST                                         14682.
                                              ----------
SUBTOTAL                                         104806.
LEGAL, FISCAL, ADMINISTRATIVE                      3524.
                                              ----------
SUBTOTAL                                         108330.
INTEREST DURING CONSTRUCTION                       1344.
                                              ----------
GRAND TOTAL                                      109675.
                                              ----------
                                              ----------


     QD         RI        YRS        DA        DHR       CCI        WPI
   0.100      0.060     25.000  1500.000     4.340     1.881      1.405

   BCDIN      PLOAD       HP       DCL2       TCL2     FSLAB      HEAD
  200.000    50.000     0.000     0.000      0.000     0.000     30.000

    FORK       TEMP      SLAB      CL2       DEEP      SAER       PUMP
   0.000      1.000     0.000     0.000      0.000     0.000      1.000
```

Figure B-4. Cost estimate for waste stabilization ponds, warm climate.

```
          .1 MGD     WASTE STABILIZATION POND     COLD CLIMATE     PUMPING     JAN 1974

                                               COST ESTIMATE
                                                    FOR
                                           WASTE STABILIZATION PONDS

      PROCESS COMPONENTS             DESIGN    CONSTRUCTION      CAPITAL   DEBT COST    O+M COST  TOTAL COST     EXCESS
                                   PARAMETER    COST, $          COST, $   CTS/1000    CTS/1000    CTS/1000    CAPACITY

RAW WASTEWATER PUMPING              0.10 MGD        29602.        43138.      9.245      12.262      21.508        1.00
STABILIZATION POND-NONAERATED-C     9.80 ACRES      98469.       143494.     30.753       4.644      35.397        1.00
                                               ----------    ----------  ----------  ----------  ----------
SUBTOTAL                                           128072.       186632.     39.999      16.906      56.905
ENGINEERING COST                                    20669.
LAND REQUIRED, 20.03 ACRES
LAND COST                                           30049.
                                               ----------
SUBTOTAL                                           178791.
LEGAL, FISCAL, ADMINISTRATIVE                        4964.
                                               ----------
SUBTOTAL                                           183756.
INTEREST DURING CONSTRUCTION                         2876.
                                               ----------
GRAND TOTAL                                        186632.
                                               ----------
                                               ----------


     QD          RI          YRS         DA        DHR        CCI        WPI
   0.100       0.060       25.000   1500.000      4.340      1.881      1.405

   BODIN       PLOAD         HP         DCL2       TCL2      FSLAB       HEAD
  200.000      17.000       0.000      0.000      0.000      0.000     30.000

    FORK        TEMP        SLAB        CL2        DEEP       SAER       PUMP
   0.000       0.000       0.000      0.000      0.000      0.000      1.000
```

Figure B-5. Cost estimate for waste stabilization ponds, cold climate.

```
      .1 MGD          AERATED LAGOON          PUMPING          JANUARY 1974 DOLLARS

                                                 COST ESTIMATE
                                                      FOR
                                             WASTE STABILIZATION PONDS

   PROCESS COMPONENTS               DESIGN        CONSTRUCTION      CAPITAL    DEBT COST    O+M COST   TOTAL COST     EXCESS
                                   PARAMETER      COST, $           COST, $    CTS/1000     CTS/1000    CTS/1000     CAPACITY

RAW WASTEWATER PUMPING              0.10 MGD          29602.        37566.        8.051       12.262      20.314         1.00
STABILIZATION POND-AERATED 15FT     0.98 ACRES        41056.        52100.       11.166        0.000      11.166         1.00
SURFACE AERATORS-STATIONARY        10.00 HP           33477.        42483.        9.104       18.627      27.732         1.00
CONCRETE SLAB EMBANKMENT            0.98 ACRES         7401.         9392.        2.012        0.000       2.012         1.00
CHLORINATION CONTACT BASIN        278.52 CU FT         4656.         5908.        1.266        0.000       1.266         1.00
CHLORINATION FEED SYSTEM           12.49 LB/DAY       15330.        19454.        4.169        6.419      10.589         1.00
                                                  ----------    ----------   ----------   ----------  ----------
SUBTOTAL                                             131524.       166904.       35.770       37.310      73.081
ENGINEERING COST                                      21029.
LAND REQUIRED,   4.84 ACRES
LAND COST                                              7265.
                                                  ----------
SUBTOTAL                                             159819.
LEGAL, FISCAL, ADMINISTRATIVE                          4623.
                                                  ----------
SUBTOTAL                                             164442.
INTEREST DURING CONSTRUCTION                           2462.
                                                  ----------
GRAND TOTAL                                          166904.
                                                  ----------
                                                  ----------


     QD          RI        YRS         DA         DHR         CCI        WPI
   0.100       0.060     25.000   1500.000       4.340       1.881      1.405

  BODIN       PLOAD        HP        DCL2        TCL2       FSLAB      HEAD
 200.000     170.000     10.000     15.000      30.000       1.000     30.000

   FORK        TEMP       SLAB        CL2        DEEP        SAER       PUMP
    1.000       0.000      1.000      1.000       1.000       1.000      1.000
```

Figure B-6. Cost estimate for waste stabilization ponds, pumping.

NOTE:

RESEARCH ON COLD CLIMATE WASTEWATER LAGOONS

H. J. Coutts[1]

The waste treatment research effort of the Arctic Environmental Research Laboratory has been concentrated on aerated lagoons and extended aeration processes.

Stabilization (facultative) ponds have not been extensively used in Alaska, therefore most of the performance data is from Northern Canadian lagoons.

Data on facultative lagoons north of 52° north latitude indicated winter BOD removals of 30 to 73 percent at detentions from 70 to 200 days.[2] Summer BOD removals were generally above 70 percent.

The Arctic Environmental Research Laboratory research and monitoring field effort has been concentrated on an experimental lagoon at Eielson Air Force Base, Alaska; a lagoon receiving total sewage (~ 1/3 mgd) from Ft. Greely, Alaska; a small lagoon serving 14 residential houses at Northway, Alaska; and a lagoon serving Eagle River, Alaska (~ 1/7 mgd). All the above lagoons were aerated with submerged devices. The aeration rates were high enough to prevent thermal stratification but not sufficient enough to prevent settleable solids from precipitating. The detentions varied from 12 to 30 days.

Experience with the Eielson Air Force Base aerated lagoon has shown that first year data is not a reliable indicator (at least for cold regions) of long term performance. Due to algal blooms the first winter's (startup) performance was better than the following summer. After the first year the summer performance was consistently better (than winter). Apparently it takes a while for the bottom sludge to accumulate and digest adding to effluent BOD.

Performance data for many northern aerated lagoons are summarized and plotted in Figure 1.

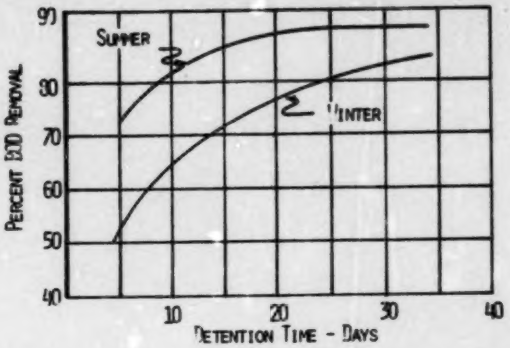


Figure 1. Average performance data on nine cold region aerated lagoon systems, Arctic Environmental Research Laboratory, August 1974.

It should be recognized that there is considerable range in the data and that many winter and summer points overlap. Figure 1 is preliminary and should not be used for design.

The data are from the four above-mentioned Alaska lagoons, three Canadian research lagoons, one North Dakota, and three Minnesota lagoons. A comprehensive report on Cold Climate Aerated Lagoons is to be published by the Arctic Environmental Research Laboratory this winter.

Recommendations by the Arctic Environmental Research Laboratory on aeration devices are applicable to all aerated lagoons, independent of temperature. The recommendation is to avoid restricted orifices when submerged aerators are used. An example of restricted orifice devices are slitted tubing or porous stones (ceramic plugs), which tend to clog very readily. The AERL Working Paper No. 17[3] quantifies the economical advantages of using coarse bubble (open) aerators versus fine bubble (restricted aerators).

---

[1] H. J. Coutts is Chemical Engineer, Arctic Environmental Research Laboratory, Environmental Protection Agency, College, Alaska.

[2] "Biological Waste Treatment in the Far North," Federal Water Quality Administration, Northwest Region, Alaska, Water Laboratory 1610, June 1970. Now Arctic Environmental Research Laboratory, Environmental Protection Agency, College, Alaska 99701.

[3] "Coarse Bubble Diffusers for Aerated Lagoons in Cold Climates," by Conrad D. Christianson, Working Paper No. 17, U.S. Environmental Protection Agency, National Environmental Research Center, Arctic Environmental Research Laboratory, College, Alaska 99701.

NOTE:

POLISHING GRAVEL AND ACTIVATED CARBON FILTER ON AERATED LAGOON EFFLUENTS

G. Hartmann[1]

Region VIII has a construction grant project for the Boxelder Sanitation District in Fort Collins, Colorado, under construction that entails the use of a polishing lagoon with a gravel and activated carbon filter at the end of an aerated lagoon system. The last week of August was the first time that the filter was put on line and to date no data are available on its performance.

The project consists of expanding and upgrading a 4-acre facultative pond with the addition of two, 2-acre aerated lagoons that can be operated in series or parallel, followed by the existing facultative lagoon that will now be used as the polishing cell in the lagoon system. Built into the discharge end of the dike of the existing lagoon is a gravel and activated carbon filter. The project was conceived and designed to meet the State of Colorado effluent standards for 1978 of 20 mg/l for BOD_5, SS, color and turbidity.

The State of Colorado was instrumental in getting this project underway and hopes to gain data from it which will enable communities in Colorado to be able to upgrade their lagoons in an effective and economical manner. The total construction cost of the filter, including the air supply, was $56,000. Seventy-five percent of the project costs was paid for with federal grant funds.

The filter itself consists of three zones of gravel followed by a zone of activated carbon. The gravel and activated carbon are placed in such a manner that the flow through each zone is horizontal, with each zone contiguous to the next.

The pond effluent first passes through a 30-foot wide zone of 1½-inch gravel, then through a 13.5-foot wide zone of 3/4-inch gravel, then through a 4.5-foot zone of pea gravel, before passing through approximately 14 feet of coarse activated carbon. Effluent may be drawn off from the filter after going through the first two gravel zones or after passing through the entire gravel and activated carbon filter. The detention time for passage of the pond effluent through the filter is quite long, being on the order of days.

The filter has been constructed with 8-inch perforated PVC air discharge pipes in the bottom of the filter. The spacing of the air discharge pipes varies with the filter medium size, with a closer pipe spacing for the smaller media. Two 15 HP blowers and motors have been included to provide for delivery of air through the filter when required.

The Boxelder Sanitation District and the consultant plan to sample the filter influent and effluent for BOD_5, SS, NH_3, NO_3, turbidity, and color. However at this time the owners and consultant have not yet decided on the extent of the sampling program. The filter has sampling tubes installed in the filter so that samples may be drawn from several different places in the filter to evaluate the performance of the filter as flow passes through.

The district and consultant plan to operate the filter in several different modes. With the placement of air diffusers in the filter, it is hoped that it will be possible to operate the filter in an aerobic or anaerobic mode. The water level in the final lagoon can be regulated so that the filter can be kept flooded or left dry if desired. Shortly the district and consultant will be developing a series of operational modes and developing a monitoring program for each specific operational mode that is used.

The EPA Region VIII Office intends to keep as fully informed as possible on the monitoring of the filter and will attempt to make these monitoring results available as they are obtained.

Owner of the Project–Boxelder Sanitation District, Fort Collins, Colorado
Consultant–M & I Engineers, Fort Collins, Colorado

---

[1] G. Hartmann is with the Environmental Protection Agency, Region 8, Denver, Colorado.

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George M. Woolwine	EPA, Region 3, Philadelphia
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Don Zollman	Montana Dept. of Health–Helena

APPENDIX

MONITORING REQUIREMENTS FOR PONDS

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, D.C. 20460

Dr. E. J. Middlebrooks
Dean, School of Engineering
Utah State University
Logan, Utah

Dear Dr. Middlebrooks:

During the recent workshop on Wastewater Treatment Ponds which your University hosted, a question was raised regarding the monitoring requirements for ponds.

The attached memorandum was obtained from our Permit Policy Branch. You will see that their guidance is that "permits should require sampling of major parameters at least once per month for small facilities". It also recognizes that some facilities will need time to achieve minimum monitoring requirements.

Also attached is a table of monitoring requirements extracted from "Permit Program Guidance for Self-monitoring and Reporting Requirements", dated October 1, 1973.

Sincerely yours,

William A. Whittington
Sanitary Engineering
Municipal Technology Branch(WH-447)

Attachments

P.G. Permit No. 14

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY

Date: February 15, 1974

SUBJECT: Self-monitoring

FROM: Michael Cook, Acting Branch Chief
Permit Policy Branch

TO: Regional Permit Branch Chiefs
and
Regional Municipal Permit Coordinators

THRU: Kenneth L. Johnson, Director
Municipal Permits & Operations Division

Concern and some confusion was expressed at our meetings January 14 and 17 with the Regions in Atlanta and Seattle about guidance from the headquarters on monitoring frequency. This memo is intended to summarize briefly our guidance on this subject, and supplement it in the few areas where it needs clarification. The self-monitoring form to be used is also discussed.

The document, *Permit Program Guidance for Self-monitoring and Reporting Requirements*, dated October 1, 1973, states that permits should require sampling of major parameters at least once per month for small facilities, and more often for larger ones. Where facilities have industrial flows, required sampling frequency is to be based on industrial monitoring guidance in the same document. The frequencies listed in the document are considered to be minimums. Regions should establish more rigid requirements where appropriate.

Some facilities will need time to achieve minimum requirements. Guidance for these cases is provided in Jack Rhett's memo of August 20, 1973 to the Regional Municipal Permit Coordinators, entitled "Use of Draft Municipal Permits"; and in his memo of November 15, 1973 to the Regional Administrator of Region VII, entitled "Monitoring Periodicity for Small Municipalities."

Permit Program Guidance for Self-monitoring and Reporting Requirements calls for a self-monitoring report on a quarterly basis from all municipal facilities. The quarterly report is to consist of three monthly summary reports completed on the EPA-designated reporting form, or a State reporting form approved by EPA for this purpose.

We have decided after consultation with the Regional Offices to adapt EPA Form 3320-1 (10-72) for use as the municipal self-monitoring reporting form. A draft of the modifications we propose to make in this form will be distributed shortly for your comment.

Table III

Municipal Wastewater Treatment Facilities
Minimum Sampling Frequency

		Effluent					
Plant Size (mgd)	Flow	BOD_5 (mg/l)	Suspended Solids (mg/l)	pH	Residual Chlorine (mg/l)[3]	Fecal Coliform (N Per 100 ml)[1,3]	Settleable Solids (ml/l)[3]
Up to 0.99	Once Each Wkday.[2]	Once per month					
1 - 4.99	Daily	Once per week					
5 - 14.99	Daily	Once per weekday[2]					
15 and greater	Daily	Once per day					

1. In smaller plants, we should accept total coliform rather than fecal coliform at this time.
2. Weekday = Monday - Friday.
3. Grab Sample.

UTAH STATE UNIVERSITY · LOGAN, UTAH 84322
COLLEGE OF ENGINEERING
UMC 41

OFFICE OF THE DEAN
801-752-4100
ext. 7801 or 7802

October 23, 1974

James D. Nelson
South Dakota Department of
Environmental Protection
Pierre, South Dakota 57501

Dear Mr. Nelson:

We are attempting to finalize the proceedings for the symposium on upgrading waste water stabilization lagoons, and only one written question has been received for inclusion in the appendices. We are still debating as to whether to include a question and answer section because of the shortage of written questions. However, I felt it only appropriate that your question be answered, and because it was directed to me, I will attempt to answer the question in the following paragraphs.

Your question was: "A two-cell stabilization pond provides 120 day winter storage and the effluent from the pond averages 50 mg/l BOD_5 and 70 mg/l suspended solids. Would you recommend the addition of a third cell or another method of treatment such as intermittent sand filtration to comply with the secondary treatment standard?"

I doubt very seriously that the addition of a third cell to the present two-cell system will improve the effluent quality significantly. I would recommend that the intermittent sand filter, rock filter, or some other addition to the present system be considered and carefully evaluated. This is the only way that you will obtain a consistent quality of effluent and meet the secondary standard.

I realize that the above answer is rather general, but to avoid being considered biased in my recommendation I feel it only fair that you consider other techniques. The intermittent sand filter used in series with the lagoon effluent passing first through a course sand with an effective size of approximately 0.7 mm, to a finer sand (effective size ~ 0.4 mm) and eventually to a sand similar to the one that we have used in our early evaluation (effective size of .17 millimeters) will provide an excellent effluent. Based upon the preliminary data that we have collected the series process will perform an entire season before plugging if the unit is loaded at a loading rate of 0.5 million gallons per acre per day. The optimum loading rate for a series type operation remains to be determined; however, we feel sure that a value greater than 0.5 million gallons per acre per day will be selected as the design value. I would recommend very strongly that a series type system be employed. If you have additional questions or wish me to elaborate further on your above question, please let me know.

Sincerely yours,

E. Joe Middlebrooks
Dean

EJM:js

AUTHOR INDEX

A

Alley, F.C., 147, 175
American Public Health Association, 36, 45, 75, 87
American Society of Civil Engineers, 47, 68, 71, 87
Amin, J. V., 157, 174
Amramy, A., 137, 172
Anderson, D. B., 157, 175
Anne, P., 156, 172
Arnon, D. I., 152, 156, 172
Assenzo, J. R., 147, 172
Attoe, O. J., 177
Aulenbach, D. B., 185, 186
Avnimelech, Y., 137, 166, 172
AWWA, 87
Ayers, A. D., 175

B

Balassa, J., 154, 177
Bancroft, P. M., 137, 172
Bardach, J. E., 188, 189
Barnette, R. M., 177
Barsom, G., 3, 4, 5, 14
Barth, E. F., 45
Batchelder, A. R., 175
Bauer, L. D., 138, 172
Baxter, S. S., 184, 186
Bear, F. E., 176
Beath, O. A., 162, 177
Beaton, J. D., 163, 172
Bendixen, T. W., 174, 177
Berger, K. C., 177
Bernstein, L., 155, 163, 172, 173
Berry, A. E., 31, 45
Bingham, F. T., 155, 159, 172
Bisbjerg, B., 162, 172
Bishop, W. B. S., 157, 172
Blair, G. Y., 178
Bott, R. F., 175
Borchardt, J. A., 31, 45, 55, 68
Bower, C. A., 178
Bowling, J. O., Jr., 174
Bradfield, R., 173, 178
Bradford, G. R., 175
Bredell, G. S., 173
Brennan, E. G., 175, 176
Brewer, R. F., 156, 172
Bronson, J. C., 176
Brown, A. L., 158, 159, 172
Brown, J. C., 172
Brown, J. W., 153, 177
Bugbee, S. L., 191

C

Calaway, W. T., 47, 51, 58, 68, 72, 87, 88
Campbell, J. D., 156, 172
Cannell, G. H., 159, 172
Carpenter, R. L., 189
Carroll, W. R., 68, 87
Champlin, R. L., 5, 14
Chang, C. W., 157, 173
Chang, S. L., 137, 173
Chapman, H. D., 153, 172, 173, 176
Chichester, H. G., 189
Chipman, E. W., 174
Christianson, C. D., 5, 14
Clark, E., 137, 173
Clark, N. A., 137, 173
Clark, R., 174
Cleary, E. J., 137, 176
Clements, H. F., 158, 173
Clesceri, N. L., 186
Cohn, M. M., 184, 186
Colby, W. G., 178
Coleman, M. S., 189
Coleman, R., 163, 173
Cooper, R. C., 176
Cooper, W. C., 154, 173
Cope, J. T., Jr., 162, 173
Corbett, E. G., 155, 173
Cram, W. H., 178
Crites, R. W., 184, 185, 186

D

Daines, R. H., 175, 176
Davis, D. E., 176
Davison, V. E., 149, 173
Day, A. D., 137, 171, 173
Dean, R. J., 184, 186
Deaner, D. G., 183, 184, 186
Deatrick, E. P., 157, 173
Delaney, T. B., Jr., 185, 186
Dennis, J. M., 137, 173
DeTurk, E. E., 137, 173
Dinges, R., 188, 189
Dixon, N., 174
Dodd, J. C., 31, 46
Doner, H. E., 173
Dornbush, J. N., 14
Drake, M., 178
Drewry, W., 173
Druger, P., 173
Dryden, F. D., 5, 6, 14
Duffer, W. R., 187
Dupuis, M., 156, 172
Dye, E. O., 137, 173

E

Eaton, F. M., 153, 154, 155, 161, 162, 163, 173
Eaton, S. V., 173
Ehlig, C. F., 155, 163, 173
Ehrlich, S., 187, 189
Eilers, R. G., 212, 213
Elgabaly, M. M., 164, 173
Eliassen, R., 137, 173
Ellis, G. H., 174
Embleton, T. W., 157, 158, 173, 174, 176
Emerson, D. L., 88
Englebert, L. E., 174
Englehard, W. E., 172
Enns, W. R., 157, 175
Environmental Protection Agency, 36, 46, 75, 87, 199, 200, 213
Ergle, D. R., 163, 173
Evans, C. A., 172

F

Falk, L. K., 177
Farrell, M. A., 176
Federation of Sewage and Industrial Wastes Association, 47, 68, 71, 87
Feinmesser, A., 137, 173
Fenn, L. B., 172
Fergus, I. F., 158, 173
Ferris, J. G., 186
Filip, D. S., 71
Fireman, M., 163, 173, 174, 177
Folkman, Y., 31, 46, 55, 68
Forbes, R. H., 156, 173
Ford, H. W., 158, 173
Foree, E. G., 43, 46
Foster, Z. C., 174
Francis, L., 178
Frear, D. E. H., 157, 174
Fruh, E. G., 45, 46
Furman, Thomas De Daussure, 47, 51, 52, 68, 72, 88

G

Gallatin, M.H., 175
Garber, M. J., 159, 172, 173
Garner, W. W., 157, 174
Gauch, H. G., 157, 174, 175
Gausman, H. W., 155, 173
Gearheart, R. A., 137
Gee, H. K., 176
Gilbert, F. A., 163,174
Gile, P. L., 174
Gissel-Nielsen, G., 162, 172
Gloyna, E. F., 147, 174
Golueke, C. G., 176
Goudey, R. F., 137, 174
Grantham, George R., 47, 51, 52, 53, 68, 72, 88
Greer, D. E., 188, 189
Grossenbecker, K., 138, 174
Guerin, F. J. A., 46
Gupta, U. C., 158, 174
Gusta, L. V., 156, 172

H

Hall, E. D., 213
Harding, R. B., 163, 173, 174, 176
Harley, C. P., 152, 174
Harlin, C. C., Jr., 185, 186
Harris, D. J., 193, 197
Harris, S. E., 71
Harward, M. E., 155, 174
Hassan, M. N., 152, 154
Hayward, M. L., 137, 174
Henderson, J. P., 189
Henry, A. K., 88
Henry, C. O., 137, 174
Herman, E. R., 147, 174
Herzik, G. R., Jr., 137, 174
Heukelekian, H., 137, 174
Hewitt, E. J., 156, 157, 174
Hill, D., 71
Hill, R. D., 137, 174
Hills, F. J., 172
Hinman, J. J. J., 163, 174
Hoeppel, R. E., 185, 186
Holmes, R. S., 159, 172, 174
Hooper, Margerat C., 157, 174
Horner, G. M., 177
Hough, G. J., 162, 174
Huguenin, J. E., 188, 189
Hunt, Henry J., 137, 174
Hunt, P. G., 185, 186
Hunter, J. G., 156, 164, 174
Hurd-Karrer, Annie M., 162, 174
Husley, A., 188, 190

I

Ives, Kenneth J., 55, 59, 60, 68

J

Jackson, W. A., 174
Jenny, H., 138, 174
Joham, H. E., 157, 174
Johnson, C. M., 152, 172
Jones, J. H., 137, 174
Jones, N. B., 69, 146, 147, 174, 175
Jones, W. W., 173, 176
Jopling, W. F., 150, 158, 174, 175
Joshi, K. G., 159, 174

K

Kabler, P. W., 166, 176
Kardus, L. T., 176
Katko, A., 175
Keaton, C. M., 177
Keffer, William J., 193, 197
Kelley, S. M., 174
Kelley, W. C., 152, 174
Kelley, W. P., 137, 148, 175
Keshen, A. S., 171, 175
Kitterle, R. A., 146, 175
Klein, S. A., 178
Klintworth, H., 157, 175
Krantz, B. A., 172
Kraus, E. J., 158, 175
Kraybill, H. R., 158, 175

L

Labanauskas, C. K., 157, 175, 176
Langille, W. M., 154, 175
Larson, G., 174
Las Virgenes Municipal Water District, 187, 190
Lawrence, A. W., 43, 46
Lehman, W. F., 176
Leone, I. A., 156, 175, 176
Leukel, W. A., 177
Levin, G. V., 31, 46
Lewis, R. F., 3
Lieber, M., 154, 175
Lieberg, G. F., 152, 175
Lindner, R. C., 152, 174
Lipman, C. B., 153, 156, 163, 175, 177
Little, G. M., 137, 175
Loneragan, J. F., 159, 175
Long, S. K., 68, 87
Lund, Z. F., 155, 175
Lunin, J., 160, 175
Lunt, O. R., 176
Lynch, K., 148, 175

M

MacKay, D. C., 174
MacKinney, G., 156, 175
Magistad, O. C., 160, 175
MaHoney, J. F., 154, 175
Marais, G. v. R., 147, 148, 149, 175
Marshall, G. R., 47, 68, 72, 75, 76, 81, 85, 88

Martin, D. M., 31, 46
Martin, J. L., 31, 37, 46
Martin, J. P., 138, 163, 175
Mason, D. D., 174
McCarty, P. L., 45, 46
McCoy, O. D., 176
McCulloch, R. B., 172
McGarry, M. G., 31, 46
McGauhey, P. H., 178
McHargue, J. S., 157, 175
McHarness, D. D., 45, 46
McKee, J. E., 52, 69, 154, 156, 162, 164, 168, 175
McKinney, R. E., 5, 14, 15
McLarney, W. O., 189
McLarty, H. R., 154, 175
McLean, G. W., 173
McMichael, W. F., 212, 213
McMurtrey, J. E., Jr., 174
McNulty, I. B., 157, 175
McQueen, Frank, 137, 175
Meenaghan, G. F., 147, 175
Meron, A., 155, 165, 175
Merrell, J. C., 137, 175
Merz, R. C., 137, 138, 175
Meyer, D. S., 157, 175
Michelson, L. F., 178
Middlebrooks, E. Joe, 47, 58, 65, 68, 69, 71, 72, 75, 76, 81, 85, 88, 137, 144, 145, 175
Miller, M. D., 177
Miller, M. P., 174
Miller, W. M., 162, 175
Millikan, C. R., 156, 176
Mishra, P. C., 153, 176
Misra, S. G., 153, 176
Mitchell, George A., 137, 176
Mitchell, R., 176
Moldenhauer, R. E., 174
Mor, E., 176
Moss, E. G., 174
Moxon, A. L., 152, 176
Myers, E. A., 176, 184, 186
Mulbarger, M., 45

N

National Technical Advisory Committee on Water Quality Criteria, 156, 176
Nesbitt, J. B., 176
Nevo, Z., 137, 172, 176
Newman, D. W., 157, 175
Norman, N. N., 166, 176

O

O'Brien, W. J., 31, 46
Oertli, J. J., 153, 172, 176
Ohki, K., 159, 177
Olgyay, V., 149, 176
Olson, E. O., 173
Olson, O. E., 176
O'Melia, C. E., 31, 45, 55, 68
Ongerth, H. J., 165, 177
Ort, J. E., 31, 46
Oswald, W. J., 146, 147, 149, 151, 176
Overman, A. R., 170, 176
Overstreet, R., 152, 174

P

Parizek, R. R., 137, 176
Parker, C. E., 5, 14
Parker, D. S., 31, 46
Parsons, T. R., 50, 69, 75, 88
Patwardham, S. D., 174
Paul, J. R., 137, 176
Pearson, G. A., 163, 172
Peech, M., 173, 178
Peterson, H. B., 158, 160, 176, 177
Peynado, A., 173
Pierce, D. M., 89
Piland, J. E., 174
Pincince, Albert B., 52, 69
Pintler, H. E., 175
Pipes, W. O., Jr., 147, 176
Porter, L. K., 152, 176
Pound, C. E., 184, 185, 186
Pratt, P. F., 158, 176
Preston, C., 152, 177
Prince, A. L., 176
Prokopovich, N. P., 188, 190

R

Rafter, George W., 137, 176
Ragotzkie, R. A., 177
Raleigh, G. J., 155, 176
Ramati, B., 176
Rebhun, M., 175
Reid, G. W., 172
Resnicky, J. W., 172
Reuther, W., 158, 159, 173, 176
Reynolds, J. H., 69, 71, 174, 175
Richards, S. J., 163, 175
Riney, W. A., 137, 176
Robins, W. R., 175
Robinson, F. E., 160, 176
Rohde, G., 138, 176
Rosenkranz, W. A., 1
Rossiter, R. C., 159, 176
Roth, J. A., 176
Rubeck, G. G., 174
Rudolfs, Willem, 137, 166, 176, 177
Rust, A., 189
Ryan, W., 173
Rytler, J. H., 188, 189, 190

S

Salisbury, P. J., 178
Schroeder, H. A., 154, 177
Schwartz, W. A., 177
Scofield, C. S., 163, 177
Searight, W. V., 176
Shell, Gerald L., 69, 175
Shindala, A., 31, 46
Simonds, J. O., 149, 177
Sisson, L. L., 176
Skulte, Bernard, P., 137, 177
Sless, B., 175
Smith, P. F., 173, 176, 177
Smith, R., 199, 212, 213
Snider, H. J., 158, 177
Soloranzo, L., 50, 69, 75, 88
Somers, G. F., 174
Sommer, Anna L., 163, 153, 177
Specht, A. W., 172, 177
State Water Pollution Board of California, 138, 177
Stefferud, A., 149, 177
Stern, G., 5, 6, 14
Steward, F. C., 152, 177
Stewart, B. A., 138, 177
Stewart, J. W., 31, 46
Stokes, W. E., 137, 177
Strickland, J. D. H., 50, 69, 75, 78

Sullivan, R. H., 184, 186
Supper, W. E., 176
Sutherland, F. H., 172

T

Taylor, A. W., 138, 177
Taylor, G. S., 166, 174
Tchobanoglous, G., 173
Tenny, M. W., 31, 46
Thakur, P. S., 174
Thomas, R. E., 137, 166, 177, 183, 185, 186
Thorne, D. W., 152, 158, 176, 177
Tiffin, L. O., 172
Tofflemire, T. J., 186
Trask, J. D., 137, 176
Trelease, Helen M., 162, 177
Trelease, S. F., 162, 177
Trimberger, J., 188, 190
Truog, E., 162, 174, 177
Tucker, T. C., 137, 171, 173
Tyler, R. W., 176

U

Uhte, W. R., 21
Ulrich, A., 159, 177
U.S. Geological Survey, 46
U.S. Public Health Service, 151, 177
U.S. Salinity Laboratory, 138, 177
Utah State Division of Health, 152, 177

V

Vandecaveye, S. C., 151, 177
Vanselow, A. P., 153, 162, 173, 175, 177
Vavich, M. G., 173
Vennes, J. W., 14
Vergnano, Ornella, 156, 164, 174

W

Wachs, A. M., 31, 46, 55, 68
Wadleigh, C. H., 153, 157, 174, 175, 177
Waksman, S. A., 138, 175
Wallihan, E. F., 157, 177
Walter, C. M., 191
Ward, P. C., 165, 177
Warrington, Sam L., 137, 177
Water Resources Council, 213
Weller, R., 31, 37, 46
Wells, W. N., 137, 177
Welsch, W. F., 154, 175
Werkhoven, C. H. E., 161, 163, 177, 178
Whetstone, R. R., 152, 178
Wilcox, L. V., 137, 159, 173, 178
Williams, K. T., 152, 178
Williams, T. C., 184, 186
Winkelstein, W., Jr., 174
Winneberger, J. H., 137, 178
Winsser, J., 174
Wolfe, H. W., 154, 156, 162, 164, 168, 175
Woodbridge, C. G., 154, 175
Worker, G. F., Jr., 176
WPCF, 87
Wright, C. T., 166, 178

Y

Yogt, J. E., 137, 178
York, E. T., Jr., 137, 155, 178
Younger, V. B., 176

Z

Zanoni, L. J., 158, 172, 178
Ziebell, C. D., 188, 189

SUBJECT INDEX

A

- Aerated lagoons, 132
- Aeration, 22
- Aerobic zone, 11, 20
- Algae, 1, 6, 8, 17, 18, 51, 74
  - Actinastrum, 18
  - Anabaena, 18
  - Anabaena sp., 85
  - Ankistrodesmus microactinium, 18
  - Aphanocapsa sp., 85
  - Blue green coccoid, 75
  - Carteria, 18
  - Cell count, 10
  - Chlamydomonas, 18
  - Chlamydomonas sp., 85
  - Chlorella, 18
  - Chlorogonium, 18
  - Closterium, 18
  - Euglena, 18
  - Euglenoid, 75
  - Food supplement, 6
  - Green, unicellular, 75
  - Growth, 78
  - Microcystis sp., 75
  - Navicula, 18
  - Navicula sp., 75
  - Oscillatoria, 18
  - Oscillatoria sp., 75
  - Pandorina, 18
  - Pediastrum sp., 75
  - Pennate, 75
  - Phacus, 18
  - Phormidium, 18
  - Primary, 18
  - Removal, 6, 72, 85
  - Scenedesmus, 18
  - Schoideria sp., 75
- Algae removal
  - Biological harvesting, 187
- Algal removal, 55, 75
- Alkalinity, 10, 17, 45
  - Bicarbonate, 152
  - Carbonate, 152
- Alum
  - Phosphorus removal, 13
- Aluminum, 170
- Ammonia nitrogen, 10, 17, 34, 37, 38, 39, 40, 45, 75, 80, 81, 83, 85, 126, 127, 132
  - Increase in effluent, 45
- Anabaena, 51
- Anaerobic zone, 11
- Ancillary facilities
  - Boat ramps, 25
  - Flow measurement, 25
  - Housing, 25
  - Samplers, 25
- Arsenic, 152, 170
- Arctic Research Laboratory, 5
- Arctic Environmental Research Laboratory, 227
- Attendees
  - Symposium, 231

B

- Bacteria
  - Achromobacter, 18
  - Coliform, 18
  - Flavobacterium, 18
  - Primary, 18
  - Pseudomonas, 18
  - Removal, 58
- Bacteriological analysis
  - Form, 97
- Belding, Michigan, 125, 126, 127, 129
- Beryllium, 170
- Best practicable technology, 199
- Bioassays
  - Algal, 58
- Biochemical oxygen demand, 10, 15, 34, 36, 37, 52, 53, 54, 59, 73, 75, 80, 82, 85, 97, 98, 120, 126, 127, 129, 132, 133, 191, 193
  - Removal, 4, 5
  - Soluble, 10, 37, 38, 39, 40
  - Total, 10, 37, 38, 39, 40
- Biological harvesting, 187
  - Algae, 187
  - Bivalve mollusks, 188
  - Fish, 188
  - Research needs, 189
  - Zooplankton, 188
- Biomass removal
  - Algal, 79
- Blanding, Utah, 178
- Blue Springs, Missouri, 9, 191
- Boron, 153, 170

C

- Cadmium, 154, 170
- Calcium, 154
- California, Missouri, 39, 43
- Capital costs, 216, 217, 218, 219, 221, 223, 225
  - Design capacity, 216, 217, 218, 219, 221, 223, 225
- Cash flow, 204
- Cell count,10
  - Algae, 10
- Charges
  - Engineering fee, 27, 28
  - User, 28
- Chemical oxygen demand, 10, 34, 36, 73, 75, 80, 85, 193
  - Soluble, 10, 37, 38, 39, 40
  - Total, 10, 37, 38, 39, 40
- Chlamydomonas, 51
- Chlorella, 38
- Chloride, 155
- Chlorination, 2, 11, 130, 133, 134
- Chlorine residual, 12
- Chlorophyll, 34, 37, 38, 39, 40, 194
- Chromium, 170
- Cincinnati, Ohio, 2, 3
- Clogging
  - Soil, 166
  - Soil pores, 137

- Cold climate
  - Aerated, 227
  - Lagoons, 227
- Coliform, 1, 2, 12, 18, 133, 194
  - Fecal, 5, 10, 11, 12, 98, 129, 194
  - Reductions, 18
  - Removal, 97
- Configuration, 23
- Construction
  - Considerations, 26
  - Contract requirements, 26
  - Drawings, 26
  - Equipment lists, 26
  - Procedures, 21
- Construction cost index
  - Sewers, 202
- Continuous cash flow, 204, 205
- Contract requirements, 26
- Copper, 156
- Corinne, Utah, 9, 10
- Costs, 3, 43, 65, 66, 67, 199, 200, 202
- Cost-effectiveness analysis, 199
- Cost estimate
  - Lagoons, 224, 225
- Crop irrigation, 6, 13
  - Lagoon effluent, 183
- Crop usage
  - Lagoon effluent irrigation, 171
- Crops
  - Effects of lagoon effluent, 137

D

- Daphnia, 37, 51
- Denitrification, 201
- Design
  - Ancillary facilities, 25
  - Configuration, 23
  - Criteria, 19
  - Dike construction, 25
  - Features, 89
  - Housing, 25
  - Infiltration-percolation systems, 185
  - Inlet, 24
  - Intermittent sand filters, 47, 74
  - Irrigation, 184
  - Lagoon, 89, 145
  - Liners, 73
  - Media, 49
  - Operation, 27
  - Outlet, 24
  - Overland-flow, 185
  - Parameters, 90
  - Population projections, 29
  - Recirculation, 23
  - Rock filter, 32, 33, 44
  - Scum control, 24
- Design capacity, 216, 217, 218, 219, 221, 223
  - Capital costs, 216, 217, 218, 219, 221, 223
- Detention time, 36
- Diatoms, 51
- Dike construction, 25
- Discharge
  - Control, 91
  - Controlled, 134
  - Patterns, 92
  - Report, 99
- Disinfection, 1, 2, 11, 14
  - Chlorination, 130
  - Lagoon, 12
- Dissolved oxygen, 75, 85, 126, 127
- Drawdown, 19
- Drinking water
  - Livestock, 181

E

- Economics
  - Lagoon construction, 21
- Effective size, 86
- Effluent
  - Algal laden, 1, 6
  - BOD, 4
  - Crop irrigation, 183
  - Disinfection, 1, 2
  - Lagoon, 1, 6, 13, 169
  - Land application, 1, 13
  - Suspended solids, 4
- Effluent quality, 77, 97, 125, 132, 177, 178, 179, 180, 192
  - Seasonal trends, 114
  - Variation with time, 80
- Endogenous respiration, 16, 17
- Environmental Protection Agency, 1, 2, 3, 6, 8, 9, 13, 15, 36, 87, 191, 207, 227, 229, 234
- Eudora, Kansas, 7, 9, 31, 32
- Euglena, 51
- Eutrophication, 6
- Evaporation, 19, 20

F

- Fargo, North Dakota, 132
- Filters
  - Gravel-activated carbon, 229
  - Intermittent, 1, 6, 11, 13, 47, 49, 60, 61, 62, 71, 72, 82, 85, 86
  - Rock, 1, 6, 13, 31, 32, 33, 35, 37, 40, 44
- Flow rates, 8
- Fluorescence, 56
- Fluorine, 156, 170
- Future worth, 204

G

- Gravel-activated carbon filter, 229
- Genoa, Ohio, 12, 13

H

- Harvesting
  - Algae, 187
  - Biological, 187
  - Bivalve mollusks, 188
  - Fish, 188
  - Research needs, 189
  - Zooplankton, 187
- Hydraulic loading rate, 36, 48, 50, 52, 54, 55, 56, 57, 61, 72, 73, 79, 81, 82, 83, 84, 85, 92
- Hydrogen sulfide, 43

I

- Illinois, 130
- Infiltration-percolation, 184, 185
- Inlets, 24
- Inspection, 27
- Interest charges, 208
- Intermittent sand filters, 1, 6, 11, 47, 71, 72, 74
  - Bacterial removal, 57
  - Cleaning, 73, 75
  - Cost, 65, 66, 67
  - Design, 47
  - Evaluation, 65
  - History, 47, 71

    Hydraulic loading, 48
    Laboratory, 47, 72
    Performance, 82, 85
    Period of operation, 60, 61, 62, 63, 75
    Pilot, 47, 72
    Plugging, 59
    Series, 86
    Solids removal, 56, 57
    Underdrain, 48
Intermittent sand filtration, 13
Iron, 157, 170
Irrigation, 151, 185
    Case history, 171
    Crop usage, 171
    Effluent use, 130
    Lagoon effluent, 137
    Land required, 170, 171
    Public health aspects, 165
    State standards, 165

K

Kilmichael, Mississippi, 9, 10

L

Lagoons
    Aerated, 1, 13, 14, 15, 18, 22, 132, 223, 227
    Area, 22
    Cold climate, 227
    Configuration, 23
    Continuous discharge, 8
    Design, 2, 3, 7, 8, 15, 16, 19, 21, 22, 71, 89, 145
    Design parameters, 90
    Dike construction, 25
    Disinfection, 11, 14
    Drawdown, 19, 89
    Economics, 145,146,191,220,221,222,223,224,225
    Facultative, 3, 4, 7, 8, 10, 12, 14, 22
    Flow rates, 1, 8
    Inlets, 24
    Level of treatment, 1, 22
    Maintenance, 1, 3, 21, 27
    Microbiology, 18
    Minimum sampling, 235
    Monitoring requirements, 233, 235
    Multi-cells, 7, 8, 10, 22, 120, 129, 134
    Nitrification, 13, 14
    Non-discharge, 1
    Number in use, 3, 71
    Operation, 21, 22, 90
    Operational problems, 6
    Outlets, 24
    Oxidation, 1, 15, 18
    Parallel, 8, 22
    Performance, 3, 4, 5, 8, 11, 22, 89, 100, 169, 192, 194, 195
    Recirculation, 23
    Series, 8, 22
    Short circuiting, 3
    Size, 1, 3, 8, 16, 19
    Soil mantle disposal, 137
    Supplemental treatment, 1
    Surface area, 90
    Suspended solids removal, 4, 5, 10
    Tertiary, 22, 49, 131, 134
    Upgrading, 1, 31, 72, 236
    Wastewater treatment, 15, 71
    Weed control, 6
Land application, 1, 6, 183, 185, 186
Land disposal, 13
Land use
    Planning, 147
    Site selection, 147
Lead, 157, 170
Light
    Effect on operation,77
Liners, 73
Lithium, 170
Livestock
    Drinking water quality, 181
Loading
    Hydraulic, 36, 48, 50, 52, 54, 55, 56, 57, 61, 72, 73, 79, 81, 82, 83, 84, 85, 92
    Organic, 3, 16, 92
Loading rates, 22
    Organic, 8
Logan, Utah, 7, 11, 49, 73

M

MPN, 12, 123
Magnesium, 157
Manganese, 157, 170
Maps
    USA, 9
Michigan, 100, 124
Microbiology, 18
Molybdenum, 170
Monitoring
    Requirements, 233
Municipal bonds, 207
Municipal wastewater treatment
    Needs, 210, 211

N

Nickel, 170
Nitrate nitrogen, 10, 12, 75, 80, 81, 84, 85, 126, 127, 194
Nitrification, 11, 13, 14, 81, 126, 134
Nitrite nitrogen, 10, 75, 80, 81, 84, 85
Nitrogen, 51, 158
    Ammonia, 10, 12, 17, 34, 37, 38, 39, 40, 52, 53, 75, 80, 83, 85, 126, 127, 128, 132
    Nitrate, 10, 12, 52, 53, 75, 80, 84, 85, 126, 127, 194
    Nitrite, 10, 12, 75, 80, 84, 85
North Dakota, 124

O

O&M manuals, 27
Odors, 3, 6
Operation, 27
    Discharge, 92
    Intermittent sand filters, 75, 76, 77
    Lagoon, 90
    Period of, 60, 61, 62, 63, 72, 77
    Records, 91, 93
Operator training, 28
Organic loading, 16
Outlets, 24
Overland-flow, 185
    Design, 185
Oxygen, 17, 18

P

pH, 5, 18, 37, 38, 49, 75, 85, 126, 132, 194
Performance, 3, 4, 5, 8, 192
    Evaluation, 8, 10, 28
    Intermittent sand filter, 82, 85
    Lagoons, 89, 97, 100, 123, 125, 127, 169
    Multi-cell lagoons, 120
    Seasonal, 127, 128, 133
    Seasonal trends, 114
    Soil mantle systems, 137

Peterborough, New Hampshire, 9, 11
Phosphorus, 34, 37, 38, 39, 40, 158
  Alum, 13
  Orthophosphate, 75, 81, 126
  Removal, 13
  Sludge, 13
  Total, 75, 81, 126, 129, 132
Photosynthesis, 23
Phytoplankton, 195
Planning
  Examples, 149, 150
  Lack of, 5
  Techniques, 149, 150
Population projections, 29
Potassium, 159
Present value, 204
Protozoa
  Colpidium, 18
  Euplotes, 18
  Glaucoma, 18
  Paramecium, 18
  Vorticella, 18

R

Recirculation, 23
Reed City, Michigan, 133
Regions
  Great Lakes Basin, 4
  Middle Atlantic, 4
  Missouri Basin, 4
  Northeast, 4
  Northwest, 4
  Ohio Basin, 4
  South Central, 4
  Southeast, 4
  Southwest, 4
Reports
  Operating, 93
Research and development, 2
Retention period, 19
Rock filter, 13
  Ammonia nitrogen increase, 45
  Costs, 43
  Design, 44
  Gradation, 43
  Hydrogen sulfide production, 43
  Solids buildup, 40
Roosevelt, Utah, 178
Rotating biological contractor, 12
Rotifers, 18

S

Salinity, 160, 161
Sampling, 8, 73
  Minimum frequency, 235
Sand
  Effective size, 49, 50, 72, 86
Scum control, 24
Seasonal variation
  Performance, 127
Secondary treatment
  Standards, 1, 5, 8
Selenium, 162, 170
Settleable solids, 20
Sewers
  Pressure, 2
  Vacuum, 2
Short circuiting, 3
Sieve analysis, 73
Silica, 162
Silver, 162
Sludge
  Buildup, 13
Sludge handling, 214
Sodium, 162
Soil
  Analysis, 143
  Bridging, 138
  Clogging, 166
  Permeability, 138
  Sedimentation, 138
  Straining, 138
Soil mantle treatment, 137, 183
Solids buildup, 40
South Dakota, 236
St. Charles, Maryland, 13
Storage
  Long range, 134
Storm water, 201
Sulfate, 163, 164
Surveillance, 3
Suspended solids, 10
  Removal, 4, 5, 187
  Total, 12, 34, 36, 37, 38, 39, 40, 48, 50, 55, 56, 57, 75, 76, 77, 78, 80, 85, 97, 120, 126, 129, 132, 133, 137
  Volatile, 12, 34, 36, 37, 38, 39, 40, 50, 55, 56, 57, 75, 76, 78, 80, 82, 85, 120
Symposium
  Membership, 231
Synthesis, 16, 17

T

Technology transfer
  EPA, 6
Temperature, 19, 36, 195
Tertiary lagoons, 131
Toxicity
  Plants, 168

U

University of Kansas, 17
Upgrading, 31, 32, 37
  Algae removal, 6
  Wastewater ponds, 1, 2, 5
Utah
  Water quality standards, 144, 145
Utah State University, 2, 11, 48, 72, 86, 141

V

Vanodium, 170
Volatile suspended solids, 10

W

Wastewater treatment, 15
  Biological concepts, 16
Water pollution, 6
Water pollution control
  Cost-effectiveness, 199
Water quality
  Factors, 167
Weeds
  Control, 6

Z

Zinc, 163, 170
Zooplankton, 75, 187
  Algae harvesting, 187